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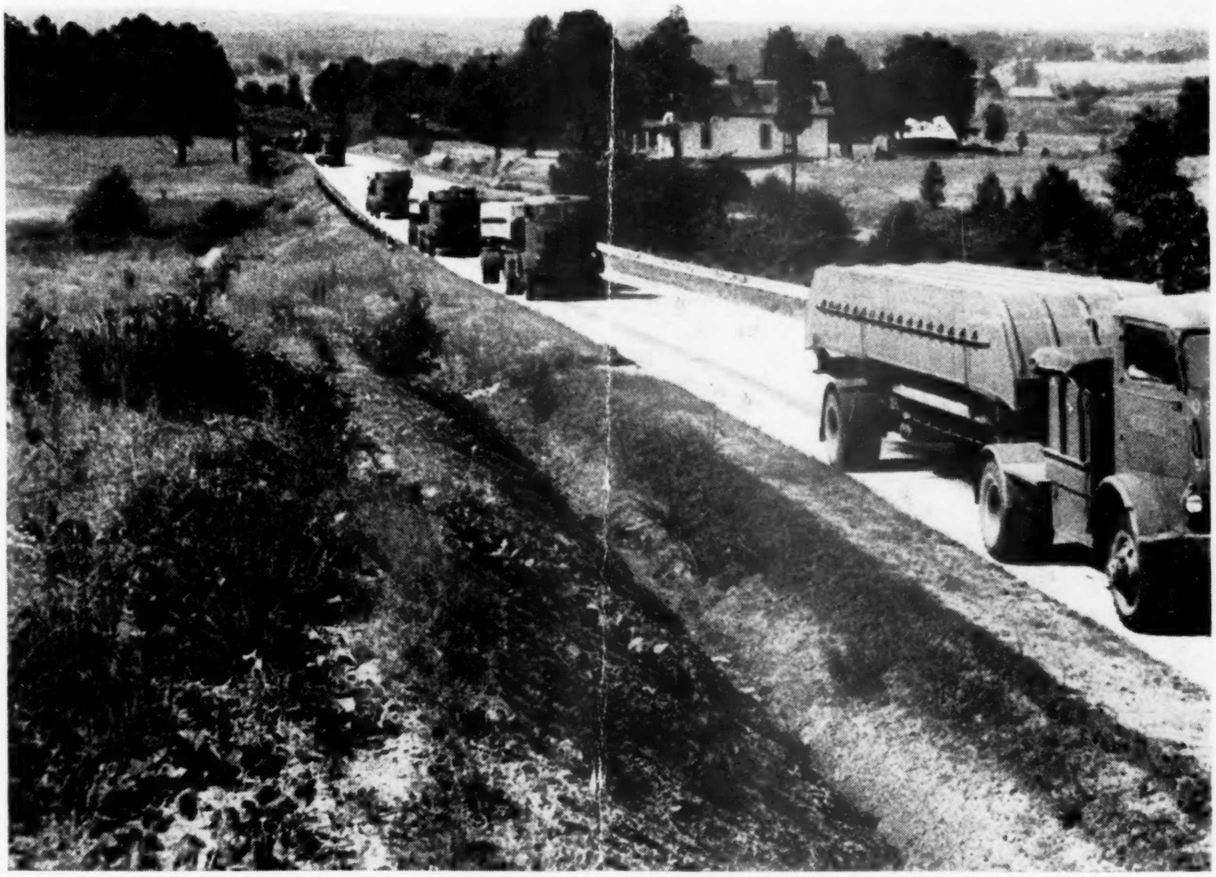

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LONGITUDINAL CRACKING IN CONCRETE PAVEMENTS

A REPORT ON A STUDY OF CRACKING IN KENTUCKY, INDIANA, AND ILLINOIS

By the Division of Physical Research, Public Roads Administration

Reported by EARL C. SUTHERLAND, Senior Highway Engineer

SERIOUS LONGITUDINAL CRACKING has occurred in a number of the concrete pavements constructed during the war emergency, some of the most serious of which has been found in Kentucky, Indiana, and Illinois. The pavements on which this cracking occurred are on strategic highways that carried a large amount of heavy war traffic. It was suspected that this cracking might have been caused by the exceptionally heavy traffic or that it might be the result of certain changes in the design of the 1942–43 pavements which it had been necessary to make because of war restrictions.

The cracking was first noted approximately 3 months after the pavements were opened to traffic and each of the three States made independent investigations of the probable cause in the spring or early summer of 1943. It was found that the longitudinal cracking had occurred in panels where there had been a delay in fracturing or actual functioning of the weakened-plane longitudinal joint.

A cooperative investigation to study this condition was started during the summer of 1943 by the Public Roads Administration and the State Highway Departments of Kentucky, Indiana, and Illinois. As a result, two surveys have been made of the pavements included in the investigation, the first during the summer of 1943 when they were approximately 9 months old and the second during the summer of 1944.[1]

This report includes the material collected in the independent investigations by the States and in the cooperative investigation.

It was known from the investigations made independently by the States that serious longitudinal cracking had occurred only in the pavements constructed in 1942. However, it was decided to make a general inspection of a limited number of older pavements of standard design that had been subjected to the same traffic as the 1942 pavements. An inspection was made also of a limited number of 1943 pavements to determine whether there was a delay in the fracturing of the longitudinal joint. The 1943 pavements were parts of 1942 projects which were not completed until the spring of 1943.

The pavements studied in the cooperative investigation together with certain design data and other information are presented in tables 1, 2, and 3.

The Kentucky pavements are located on route U S 31–W between Upton and Kosmosdale. Indiana pavement SN–FA 13–B (2) is located on route U S 40 near Greenfield while Indiana pavements SN–FA 74–A (2) A (3) B (5) and 74–D (5) H (2) are located on route U S 52 between Lebanon and LaFayette. Illinois pavements DA–WI 4–A (1), DA–WI 4–C 4–D, and SN–A–FA 25 (7) (2) are located on U S 66 Alt. a short distance south of Joliet while pavement FA 133–D is located on State route 125 a short distance west of Springfield.

WAR CONDITIONS NECESSITATED CHANGES IN DESIGN

A reasonably large range in thickness of pavements is represented in the entire group of projects. None of the 1942–43 pavements was reinforced and the dummy center joint was generally used. The width of the ribbon in the dummy center joint was .25, .33, and .30 of the center depth of the pavement in Kentucky, Indiana, and Illinois, respectively. The depth of the ribbon below the surface varied from slightly below the surface to as much as one-half inch or more.

The specifications for the aggregates, the concrete mixture, and the placing of the concrete for the 1942–43 pavements were the same as those which had been used by the different States during the previous several years. Also the strengths of the concrete, as determined from control specimens, were normal.

The important changes in the design of the 1942–43 pavements as compared to the standard design used previously were:

Kentucky

1. A dummy longitudinal joint, without tie bars, was substituted for one containing a deformed metal plate with tie bars.
2. Distributed reinforcement was eliminated and the cross section was changed from a 9–7–9-inch thickened-edge section to one of 8-inch uniform or a 9–8–9-inch thickened-edge section.
3. Load-transfer devices were eliminated at all transverse joints.

Indiana

1. Distributed reinforcement was eliminated.
2. The spacing of the contraction joints was reduced from 40 to 20 feet.
3. Load-transfer devices at the contraction joints were eliminated.

Indiana has used the dummy longitudinal joint for a number of years.

Illinois

1. A dummy longitudinal joint was substituted for a joint containing a deformed metal plate.
2. Distributed reinforcement in the pavement and load-transfer devices in the transverse joints were eliminated.
3. The thickness of the cross section of the 1942 pavements was greater than that used previously.

No part of the 1942–43 pavements was opened to traffic prematurely. The Indiana pavements were

[1] Those participating in the field survey included G. L. Logan, E. L. Lyons, T. R. Thomas, and Everette Gordon of the Kentucky, Department of Highways; R. F. Berns of the Indiana State Highway Commission; F. W. Fullenwider, R. W. Gerling, O. Larsen, E. R. Clemmons, and John Burke of the Illinois Division of Highways; and Mack Galbreath, W. R. Woolley, E. C. Sutherland, D. C. Brooks, P. M. Cassidy, and G. R. Harr of the Public Roads Administration.

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TABLE 1.—*Design data on various projects included in the investigation in Kentucky*

Federal-aid project No.	US Route No.	Year built	Length (approximate)	Width	Cross section	Reinforcement	Type of longitudinal joint	Transverse joints: Expansion: Spacing	Transverse joints: Expansion: Width	Transverse joints: Expansion: Load transfer	Transverse joints: Contraction: Spacing [1]	Transverse joints: Contraction: Load transfer	Type of coarse aggregate	Method of curing	Average strength of concrete: Compressive 28 days	Average strength of concrete: Transverse 7-14 days
			Miles	*Feet*	*Inches*			*Feet*	*Inches*		*Feet*				*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$
SN-FA 169-E (1)	31 W	1942-43	4.4 .6	22 40	9-8-9 8 Unif.	None	Dummy ⅛- x 2-inch ribbon.	120	1	None	20	None	Limestone	Membrane (Aquastatic).	3,720	
SN-FA 169-C (2)	31 W	1942-43	.6	22	9-8-9	None	Butt construction joint.	120	1	None	20	None	do	Burlap and Sisal Kraft paper 72 hours.	3,250	
SN-FA 169-F (2)	31 W	1942	7.0 .2	22 40	9-8-9 8 Unif.	None	Dummy ⅛- x 2-inch ribbon.	120	1	None	20	None	do	do	3,530	620
DA-WR 3	31 W	1942-43	10.2 .5	22 40	8 Unif. 8 Unif.	None	do	120	1	None	20	None	do	do	3,730	650
DA-WR 1	31 W 60	1942-43	3.9	22 and 44	9-8-9	None	Dummy ⅛- x 2-inch ribbon, tie bars 4 ft. c-c.	500	1	None	20	None	Gravel (Ohio River).	Membrane (Truecure).	3,765	690
FA-79-B (2)	31 W	1937	2.7	20	9-6½-9	43-lb. wire mesh.	Metal center strip, tie bars 4 ft. c-c.	90	¾	¾-inch dowels 12 in. c-c.	30	¾-inch dowels 12 in. c-c.	Limestone	Burlap and straw 7 days.	3,910	
FA-79-D (2)	31 W 60	1940	8.6	22 and 33	9-7-9	41-lb. wire mesh.	Metal center strip, tie bars 4 ft. c-c.	120	1	Yes	60	Yes	do	Burlap and Sisal Kraft paper 72 hours.	4,100	

[1] Spacing of joints including expansion joints.

TABLE 2.—*Design data on various projects included in the investigation in Indiana*

Federal-aid project No.	US Route No.	Year built	Length (approximate)	Width	Cross section	Reinforcement	Type of longitudinal joint	Transverse joints: Expansion: Spacing	Transverse joints: Expansion: Width	Transverse joints: Expansion: Load transfer	Transverse joints: Contraction: Spacing [1]	Transverse joints: Contraction: Load transfer	Type of coarse aggregate	Method of curing [2]	Average compressive strength 6 months
			Miles	*Feet*	*Inches*			*Feet*	*Inches*		*Feet*				*Lbs. in.*$^{-2}$
SN-FA 13-B(2)	40	1942	6.5	22	9-7-9	None	Dummy ⅛ x 2½-inch ribbon, tie bars 5 ft. c-c	120	¾	Dowels 12 inch c-c	20	None	Gravel	Burlap and straw.	5,210
SN-FA 74-A(2), A(3), B(5)	52	1942	8.4	22	9-7-9	None	do	120	¾	Dowels 12 inch c-c. and Translode.	20	None	do	do	6,150
SN-FA 74-D(5), H(2)	52	1942	8.3	22	9-7-9	None	do	120	¾	do	20	None	Mixture stone and gravel.	do	5,400

[1] Spacing of joints including expansion joints.
[2] Wet burlap 1 day and straw 6 days.

TABLE 3.—*Design data on various projects included in the investigation in Illinois*

Federal-aid project No.	US Route No.	Year built	Length (approximate)	Width	Cross section	Reinforcement	Type of longitudinal joint	Traverse joints: Expansion: Spacing	Traverse joints: Expansion: Width	Traverse joints: Expansion: Load transfer	Traverse joints: Contraction: Spacing [1]	Traverse joints: Contraction: Load transfer	Type of coarse aggregate	Method of curing [2]	Average transverse bending strength at 14 days
			Miles	*Feet*	*Inches*			*Feet*	*Inches*		*Feet*				*Lbs.in.*$^{-2}$
DA-WI 4-A(1)	66 (Alt.)	1942	4.6	22	10 Unif	None	Dummy 1/8- x 3-inch ribbon, tie bars 30 in. c-c.	120	¾	None	20	None	Gravel	Paper and straw.	800
DA-WI 4-C 4-D	66 (Alt.)	1942	4.3	22	10 Unif	None	do	120	¾	None	20	None	Crushed gravel	do	890
SN-A-FA 25 (7) (2)	66 (Alt.)	1942	5.6	22	10 Unif	None	do	120	¾	None	20	None	Gravel	do	670
FA 133-D	125	1942	3.6	22	10 Unif	None	do	120	¾	None	20	None	Crushed gravel	Paper	780

[1] Spacing of joints including expansion joints.
[2] Where paper curing was used the period was 72 hours. Straw curing, indefinite period, was used on late season construction.

TABLE 4.—*Summary of subgrade data for Indiana projects*

Project and station	Coarse sand	Fine sand	Silt	Clay	Colloids	Liquid limit	Plasticity index
Project SN-FA 13-B(2):	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
584+00	10	19	44	13	4	37	9
620+00	12	23	38	14	6	26	8
646+00	8	20	52	15	4	33	7
702+00	6	31	38	17	5	30	7
765+00	7	28	44	16	4	26	6
822+00	7	27	42	16	6	24	4
843+00	11	19	41	19	8	37	14
855+00	9	24	29	19	10	21	6
Project SN-FA 74-A(2) A(3) B(5):							
1400+00	4	20	56	13	6	30	9
1456+00	3	17	48	23	8	28	7
1510+00	7	18	50	13	6	28	7
1532+00	3	35	37	20	5	30	12
1560+00	3	14	59	15	9	27	6
1615+00	5	25	47	15	6	32	10
1655+00	6	21	46	21	5	28	5
1703+00	5	25	38	21	8	28	4
1715+00	4	13	51	20	11	39	12
1744+00	3	18	56	17	6	27	3
1774+00	4	19	48	19	9	32	9
1806+00	4	16	55	15	11	37	6
1847+00	7	16	54	11	9	33	8
Project SN-FA 74-D(5) H(2):							
330+00	2	22	35	28	12	28	5
380+00	10	21	41	15	11	40	10
400+00	2	22	50	19	7	30	7
454+00	7	27	42	16	7	32	10
470+00	5	17	46	12	19	33	10
530+00	5	18	54	15	7	33	8
610+00	3	20	54	18	5	31	8
675+00	4	24	48	18	5	32	10

closed to traffic for 21 days after the concrete was placed or until the control specimens showed a transverse bending strength of 550 pounds per square inch. The Illinois pavements were closed to traffic until the control specimens showed a transverse bending strength of 650 pounds per square inch.

SUBGRADE AND DRAINAGE CONDITIONS WERE GENERALLY GOOD

It was not possible for the Kentucky authorities to make a special subgrade study in connection with this investigation. However, it is known from the investigations made prior to construction that there is nothing unusual about the subgrade under the pavement where the longitudinal cracking occurred. The subgrade for these pavements was constructed under the same specifications and with the same care as has been customary in the past. The surface drainage was excellent on all of the pavements investigated in this State.

Certain physical properties of the soils in the subgrades of the various projects investigated in Indiana are shown in table 4. The samples from which these data were obtained were taken at depths of from 0 to 18 inches below the natural ground elevation prior to grading. The soil was of the same type on all three pavements and it was very uniform over the full length of each pavement. Samples taken at depths greater than 18 inches showed the same characteristics as those taken at a less depth.

The type of subgrade under the pavements studied is very common in Indiana and the soils experts of the State could find nothing unusual about the subgrade where longitudinal cracking occurred. The subgrade was constructed under the same specifications and with the same care as had been customary in the past. The surface drainage was excellent on all the pavements investigated. Subgrade drains had been placed at all wet areas found during construction.

On Indiana pavement SN-FA 74-D (5) H (2) a 6- to 8-inch sand-clay base was placed on the northbound lane between the following stations: 409+48–465+50, 493+00–650+00, 680+00–698+00.

TABLE 5.—*Summary of subgrade data for Illinois projects*

Project and station	Sand	Silt	Clay	Colloids	Liquid limit	Plasticity index
Project DA-WI 4-A (1):	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
14+00	56	27	17	5	35	12
69+00	13	58	29	10	35	11
113+00	14	61	25	6	56	25
160+00	12	62	26	7	38	12
209+00	18	65	17	3	44	15
243+00	20	60	20	3	45	19
Project DA-WI 4C, 4D:						
26+00	12	61	26	4	49	16
50+00	14	59	26	7	38	12
83+00	13	58	29	10	35	11
134+00	16	55	29	11	39	14
178+00	13	48	39	12	55	30
210+00	15	46	39	18	41	17
230+00	14	40	46	----------	36	17
295+00	16	59	25	5	46	14
308+00	16	61	22	8	31	9
342+00	9	59	32	8	47	16
359+00	46	36	18	4	30	10
Project SN-A-FA 25 (7) (2):						
250+00	7	71	22	6	41	16
337+00	36	45	18	4	35	13
352+00	20	38	42	16	57	34
Project FA 133-D:						
1+00	6	70	24	10	33	12
16+00	5	60	35	14	42	18
29+00	4	60	36	16	43	22
51+00	6	59	35	17	46	26
127+04	13	70	17	6	27	5

The physical properties of the subgrades on the pavements investigated in Illinois are shown in table 5. These data were obtained from samples taken at depths of from 0 to 18 inches below the natural ground elevation prior to grading. The subgrades for the 1942-43 pavements were constructed under the same specifications used during the previous several years. The surface drainage on all pavements was excellent.

On Illinois pavement DA-WI-4-A (1) a 6-inch crushed-stone base was placed under the pavement. A typical gradation of the material used in the base is as follows:

	Percent
Passing 1-inch sieve	100
Passing ½-inch sieve	78
Passing No. 4 sieve	43
Passing No. 8 sieve	30
Passing No. 16 sieve	20
Passing No. 200 sieve	10

No base was placed under the pavement on Illinois pavement DA-WI-4 C, 4-D. Part of the Illinois pavement project SN-A-FA 25 (7) (2) was laid directly over an old concrete pavement. The old pavement was 18 feet wide while the new pavement is 22 feet wide. The 2-foot overhang on each side was placed on a 6-inch gravel base, a typical gradation of which follows:

	Percent
Passing 3-inch sieve	100
Passing No. 4 sieve	48
Passing No. 50 sieve	12
Passing No. 200 sieve	6

PAVEMENTS WERE SUBJECTED TO HEAVY TRAFFIC

Traffic data on the pavements studied in the three States are shown in table 6.

Kentucky.—The pavements in this State on which the longitudinal cracking developed are located in the vicinity of Fort Knox and carry a large amount of heavy truck traffic. While not indicated in the table, Kentucky pavements DA-WR 3, FA 79-B (2), FA

TABLE 6.—*Traffic data on pavements studied*

Kentucky				Indiana		Illinois				
Project	Average daily traffic in 1942	Project	Average daily traffic in 1942	Project	Average daily traffic in 1943	Project	Average daily traffic in 1943	Commercial traffic— Time of survey	Average daily	Heavy daily
SN-FA 169-E (1)	875	FA 79-B (2)	6,300	SN-FA 13-B (2)	1,690	DA-WI 4-A (1)	3,520	1942	650	65
SN-FA 169-C (2)	1,475	FA 79-D (2)	6,500	SN-FA 74-A (2) A (3), B (5).	1,550	DA-WI 4-C, 4-D	3,520	1942	650	65
SN-FA 169-F (2)	2,000	DA-WR 1	8,000	SN-FA 74-D (5) H (2)	1,550	SN-A-FA 25-(7) (2)		1941	870	550
DA-WR 3	7,250					FA 133-D		1941	400	40

TABLE 7.—*Summary of crack survey data on the Kentucky projects*

Project and station	Panels in which longitudinal cracking occurred: 1943		1944	
	Number	*Percent*	*Number*	*Percent*
Project SN-FA 169-F (2):				
829—840	1	2	2	4
840—850	1	2	4	8
850—860	2	4	3	6
860—870	0	0	0	0
870—880	0	0	0	0
880—890	3	6	4	8
890—900	0	0	3	6
900—910	0	0	0	0
910—920	0	0	6	12
920—930	1	2	1	2
930—940	2	4	5	10
940—950	2	4	5	10
950—955	14	56	14	56
955—960	1	4	1	4
960—965	3	12	3	12
965—970	6	24	6	24
970—975	0	0	0	0
975—980	5	20	5	20
980—985	1	4	1	4
985—990	6	24	6	24
990—995	4	16	8	32
995—1,000	5	20	5	20
1,000—1,001+70	1	12	1	12
1,001+70—1,008+25	3	5	3	5
Project DA-WR 3:				
0—28 [1] (two left lanes)	0	0	0	0
0—28 (two right lanes)	42	30	44	31
28—81	222	84	222	84
81—134	150	57	150	57
134—186	46	17	50	19
186—239	70	26	80	30
239—292	66	25	70	26
292—345	63	24	63	24
345 (2 miles) to 450	145	27	145	27
450—503	2	1	6	2
503—556	6	2	6	2
557—567 [2]	0	0	0	0

[1] Four-lane pavement between stations 0 and 28.
[2] A separate 11-foot lane between stations 557 and 567.

79-D(2), DA-WR 1, and part of SN-FA 169-F have been used for testing tanks from Fort Knox. These tanks weigh approximately 32 tons and travel at 20 to 25 miles per hour. Until about 6 months before the survey the tank treads were equipped with rubber pads but after that time many of them were equipped with steel lugs.

The war emergency load limit in Kentucky was 28,000 pounds gross load with a maximum axle load of 16,000 pounds.

Indiana.—In a survey on Indiana pavement SN-FA 13-B (2), taken over five 8-hour days, it was found that 24.5 percent of the traffic was commercial and an average of 29 trucks, per 8-hour day, exceeded 30,000 pounds gross weight. Two overloaded trucks, one weighing 60,000 pounds and the other 57,000 pounds, were noted during the 5-day period.

In an 8-hour survey, August 1943, on State route 52 between Indiana pavements SN-FA 74-A (2) A (3) B (5) and SN-FA 74-D (5) H (2) it was found that 29 percent of the traffic was commercial. The gross weights of the 10 heaviest trucks were: 55,000, 51,600, 47,200, 46,200, 45,400, 44,800, 44,400, 42,000, 40,200, and 40,000 pounds.

The maximum legal axle load in Indiana is 18,000 pounds. The number of overloaded trucks on the pavements during the war greatly exceeded those found in normal times.

Illinois.—The number of commercial and heavy vehicles on the various Illinois pavements are shown in table 6. The maximum axle load in this State is 16,000 pounds. The maximum gross loads for various types of vehicles were:

	Pounds
Vehicle with 2 axles	24,000
Vehicle with 3 axles	40,000
Truck and trailer combination with at least 4 axles	56,000

No army tanks used the pavements on which longitudinal cracking occurred in either Indiana or Illinois.

The ribbon placed in the pavement to form the dummy longitudinal joint was generally placed a short distance below the surface. It was found from cores drilled at the center joint that the presence or absence of a crack over the ribbon was an indication of whether or not the longitudinal joint had fractured. For this reason the presence or absence of such a crack was noted in the crack surveys. The core data will be presented later in this report.

CRACK SURVEY DATA IN KENTUCKY

Kentucky pavement SN-FA 169-E (1).—The surface studied was laid in the spring and early summer of 1943. At the time of the first survey this pavement had been subjected for a short time to heavy truck traffic but no army tanks had passed over it regularly. Longitudinal cracks were not found in the first survey, and it was estimated that the longitudinal joint had not fractured in approximately 40 percent of the panels. In the second survey three full-length and three part-length cracks were found and the center joint still had not fractured in 10 percent of the panels.

Kentucky pavement SN-FA 169-C (2).—This pavement was constructed in two separate 11-foot lanes with a butt longitudinal joint between them. It had been subjected to heavy truck traffic, but had not been used by army tanks. Longitudinal cracking had not occurred up to the time of the last survey.

Kentucky pavement SN-FA 169-F (2).—This pavement carried a large amount of heavy truck traffic throughout its full length. Between stations 950 and 1008 it had been used by a large number of medium army tanks, but few if any army tanks had used the pavement between stations 829 and 950.

The data obtained in the two surveys are summarized in table 7. The word "panel" as used in this and succeeding tables applies to the pavement width of two

lanes and the length between successive transverse joints. In the analysis of the crack survey data a crack extending three-quarters or more of the length of the panel was tabulated as a full-length crack while one of lesser length was tabulated as a part-length crack.

At the time of the first survey in 1943, a total of 2 percent of the panels contained full- or part-length longitudinal cracks between stations 829 and 950 and 15 percent between stations 950 and 1008+25. In the survey of 1944, a total of 5 percent of the panels were cracked between stations 829 and 950, and 16 percent between stations 950 and 1008+25. Fifty-two percent of the cracks in this surface were full length. In the first survey only one or two longitudinal cracks were present in panels where the center joint had fractured, but in the second survey there was a total of eight such cracks. Ten panels were found in the second survey in which the center joint had not fractured and in which no longitudinal cracks had occurred.

It is very possible that the greater amount of longitudinal cracking between stations 950 and 1008+25 as compared to that between stations 829 and 950 was due to the stresses caused by the army tanks.

Kentucky pavement DA–WR 3.—This was a four-lane pavement between stations 0 and 28. The four lanes were divided at the center with a butt longitudinal joint. The longitudinal joint at the center of the two right lanes, in the direction of stationing, was of the dummy type while that between the two left lanes was of the deformed metal plate type. The pavement studied between stations 557 and 567 was a single lane laid along the edge of an older pavement. The remainder of the pavement, on this project, was two lanes with a dummy longitudinal joint. This pavement had been subjected to heavy truck and tank traffic throughout its full length.

Table 7 shows that there was no longitudinal cracking in those parts of the pavement with a definite separation at the longitudinal joint at the time of construction; that is, in the two left lanes between stations 0 and 28 and in the single-lane construction between stations 557 and 567. Another notable condition shown by the data of table 7 is the variation in the amount of longitudinal cracking between the two ends of the pavement. There appeared to be no differences between the subgrades at the two ends that would explain this variation and the whole pavement has been subjected to the same traffic. It is probable that the variation in amount of cracking was caused by differences in weather when the different parts of the pavement were laid. Construction was started in the summer at station 556 and the pavement was not completed until very late in the fall.

There was little difference in the amount of cracking observed in the two surveys. In the 1944 survey, 63 percent of the cracks were full length, 10 panels were found with longitudinal cracks on both sides of the longitudinal joint, 16 in which longitudinal cracks had occurred where the center joint had fractured, and 15 in which no longitudinal cracks had occurred although the center joint had not fractured.

Kentucky pavement DA–WR 1.—Only one survey, that of 1943, was made on this pavement. At that time a study was made of 1.8 miles of four-lane pavement and a total of 3.2 miles of single-lane construction. The four-lane pavement was divided at the center with a butt construction joint, and a dummy longitudinal joint was placed between the two lanes on each side. The single-lane portion of the pavement was placed as widening along the two edges of an older pavement. The four-lane portion of the pavement and one-half of the single-lane construction (left lane) had been subjected to heavy truck and tank traffic for approximately 10 months prior to the survey. The other half of the single-lane pavement (right lane) was not completed until the spring of 1943 and had been subjected to the heavy traffic for a period of only approximately 2 months prior to the survey.

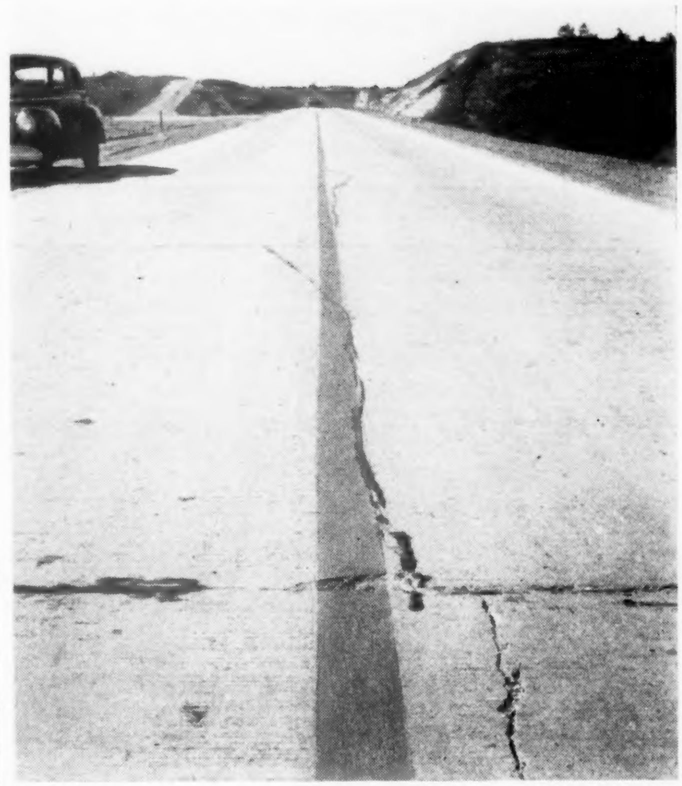

FIGURE 1.—LONGITUDINAL CRACK IN SECTION OF PAVEMENT WHERE THE CENTER JOINT HAS NOT CRACKED.

Of the 1,148 panels examined in the four-lane pavement, 220, or 19 percent, contained full- or part-length longitudinal cracks. There were no longitudinal cracks in the single-lane pavement.

Kentucky pavement FA 79–B (2) and 79–D (2).—These two pavements were of standard design and were laid between 1937 and 1941. They were included in the study because, since 1942, they had been under the same traffic as the 1942 pavements in which serious longitudinal cracking occurred. Both pavements were reinforced and the longitudinal joint was of the metal plate type. No longitudinal cracking was found in these two pavements.

DELAY IN FRACTURING CENTER JOINT AN IMPORTANT CAUSE OF CRACKING

In the 1943 survey all but one or two of the longitudinal cracks were found in panels where the dummy center joint had not fractured and therefore was not functioning as a longitudinal joint. Figure 1 shows a typical example of this condition.

Cores were drilled at the center joint in a number of panels where there was some question as to whether the

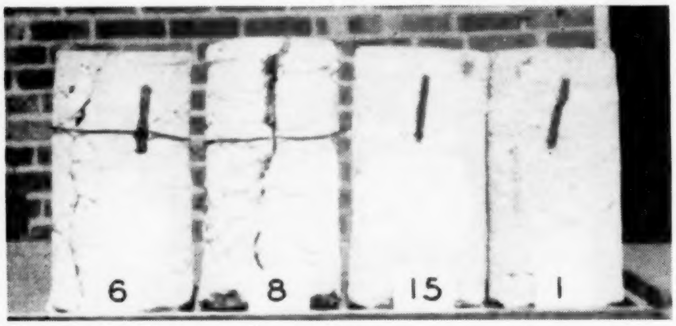

Figure 2.—Typical Cores Drilled at the Longitudinal Joint in the 1942 Kentucky Payments.

Table 8.—*Summary of the crack survey data on the Indiana projects*

Projects and stations	Time pavement was laid	Panels in which longitudinal cracking had occurred [1]			
		1943		1944	
		Number	*Percent*	*Number*	*Percent*
Project SN-FA 13-B (2):					
582—599	November	19	22	20	24
599—625	Late October	56	43	56	43
625—651	Middle October	44	34	44	34
651—677	do	68	52	68	52
677—704	Early October	12	9	12	9
704—729	Late September	1	1	3	2
729—758	do	0	0	1	1
758—782	Middle September	5	4	5	4
782—808	do	2	2	2	2
808—835	do	0	0	0	0
835—861	Early September	3	2	3	2
861—887	do	0	0	0	0
887—909	Late August	2	2	2	2
Project SN-FA 74-A (2) A (3) B (5):					
Two north-bound lanes:					
1661—1734	Early October	21	6	28	8
1734—1782	Middle October	2	1	6	2
Two south-bound lanes:					
1395—1424	Middle September	0	0	8	6
1424—1450	Early September	34	26	44	34
1450—1490	do	5	2	6	3
1490—1540	Late August	8	3	9	4
1540—1605	Middle August	14	4	19	6
1605—1650	Early August	1	0	3	1
1650—1751	July-August	1	0	0	0
1751—1834	July	0	0	1	0
1834—1861	Late September	4	3	5	4
Project SN-FA 74-D (5) H (2):					
Two south-bound lanes:					
312—410	Early August	0	0	5	1
Two north-bound lanes:					
320—407	Late July	11	3	11	3
407—538	Early July	3	0	12	2
538—618	Late June	5	1	5	1
618—671	do	4	2	5	2
671—731	Middle June	0	0	2	1

[1] Two cracks were counted in panels where cracks had occurred on each side of the longitudinal joint.

joint had fractured. Several of these typical cores are shown in figure 2. These cores show that the center joint had not fractured below the ribbon except in the one case in which a crack had appeared above the ribbon. The panel from which core 6 was taken contained a full-length longitudinal crack at a distance varying from 2 to 18 inches from the center joint, the distance being approximately 2 inches at the point where the core was drilled. The longitudinal joint had fractured in the panel from which core 8 was drilled and there was no longitudinal crack in this panel. Cores 1 and 15 were taken from panels in which full-length longitudinal cracks had occurred and, as indicated, the longitudinal joint had not fractured in these panels.

These cores show that the top edge of the ribbon was, in some cases, an inch or more below the surface of the pavement. It was thought that this might have contributed to the delay in fracturing of the joint. However, it was found in the field study that the center joint had not fractured and that longitudinal cracks had developed in many panels in which the top edge of the ribbon was near the surface.

The amount of longitudinal cracking appeared to be approximately the same in cuts and fills and followed no definite pattern either in the same or between different projects. For example, in a typical section of pavement the distance of the cracks from the center joint was as follows:

	Percent
0–12 inches	31
12–24 inches	60
24–36 inches	8
More than 36 inches	1

The part-length cracks varied from 2 to 15 feet in length and they generally broke from the longitudinal joint at some interior point and continued the remaining length of the panel. Some full-length cracks that remained fairly close to the longitudinal joint actually passed from one side of the joint to the other at two or three points within the length of the panel.

Many of the cracks had opened up badly because of the absence of reinforcement in the pavement and of tie bars in the longitudinal joint. Figure 3 shows several typical examples of cracking in the Kentucky pavements laid in 1942.

Mr. E. L. Lyons of the Kentucky Department of Highways observed that a greater amount of longitudinal cracking developed in the parts of the pavement laid late in the year than in the parts laid earlier. This condition was mentioned in the discussion of Kentucky pavement DA-WR 3, where a serious amount of longitudinal cracking developed in the parts of the pavement laid during October and November while only a small amount occurred in the parts laid earlier. (See table 7.) This was found to be true of other 1942 pavements. It appears that there is a greater delay in the fracturing of the longitudinal joint in those parts of the pavement laid late in the year and this delay increases the chances for subsequent longitudinal cracking. The possible effect of the time of the year at which the pavement was laid on the development of this form of cracking was investigated more thoroughly in Indiana and Illinois.

While this investigation was primarily a study of longitudinal cracking it was observed that in Kentucky the army tanks with the steel lug treads caused serious surface damage to the pavement and spalling at the transverse joints. There had been no transverse cracking to speak of in the 1942 pavements investigated and only a normal amount in the older pavements of standard design.

Conclusions that may be made as a result of the investigation of Kentucky pavements are:

1. There is a serious delay in fracturing of the dummy center joint as it has been constructed. This appears to be especially true of pavements laid late in the year.

2. The heavy traffic has not caused longitudinal cracking in those parts of the pavement in which there was a definite separation of the longitudinal joint at the time of construction.

3. There is a greater percentage of longitudinal cracking in the parts of the 1942 pavements used by army tanks than in the other parts.

Figure 3.—Typical Longitudinal Cracks in the 1942 Kentucky Pavements.

CRACK SURVEY DATA IN INDIANA

An intensive study was made of three pavements in Indiana laid in 1942. Serious longitudinal cracking had developed in only one of these pavements and it was first detected approximately 3 months after the pavement was opened to traffic.

The dummy center joint has been used in many pavements in this State for a number of years. Since the longitudinal cracking appeared to be associated with the dummy center joint an inspection was made of several older pavements, built with this type of center joint, in order to study the condition of the joint and determine whether or not there had been any longitudinal cracking in pavements laid prior to 1942. The crack survey data for the three 1942 pavements studied are summarized in table 8.

Indiana pavement SN–FA 13–B (2).—Little difference was found in the amount of longitudinal cracking on this surface as shown by the 1943 and 1944 surveys, the percentage of cracked panels being approximately 13 percent in each case. The amount of longitudinal cracking varied greatly between the two ends of this pavement, however. In the part laid during October and November an average of 33 percent of the panels contained full- or part-length cracks while in the part laid during August and September only a small amount of cracking appeared. Seventy-one percent of the cracks in this pavement are full length.

In the 1943 survey only one or two longitudinal cracks were found in panels having a fractured center joint. However, in the 1944 survey 56 such cracks were found. On this pavement many of the longitudinal cracks were 2 feet or more from the center joint and this may explain why the center joint fractured in so many panels in which longitudinal cracks had already occurred. At the time of the 1944 survey all of the panels in which the center joint had not fractured contained longitudinal cracks. From this it may be expected that little additional longitudinal cracking will develop in this pavement.

Several cores drilled at the center joint of panels in this pavement are shown in figure 4. The core at the left was taken from a panel with a full-length longitudinal crack approximately 3 feet from the center joint. There was no crack above the ribbon and the joint had not fractured. The center core showed evidence of cracking and there was no longitudinal crack in the panel. The panel from which the right core was taken contained a full-length crack approximately

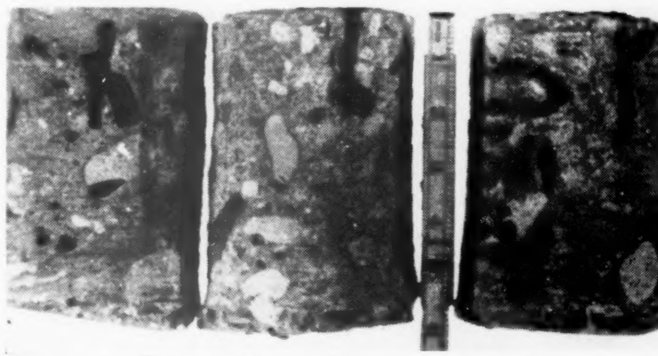

Figure 4.—Typical Cores Drilled at the Longitudinal Joint in the 1942 Indiana Pavements.

3 feet from the center joint. There was no crack above the ribbon and the joint had not fractured.

The cracks in this pavement were found at distances varying from a few inches to as much as 4 feet from the center joint. Those near the center joint were held closed by the tie bars. The cracks may be divided according to form, into two types, as follows:

1. Those which extended the full length of the panel and were generally approximately parallel to the center joint. As a rule these cracks were 12 inches or more from the center joint.

2. Those which broke from the center joint, at some intermediate point and continued through the remaining length of the panel. These cracks tended to remain fairly close to the center joint and they vary in length from approximately 2 to 15 feet. Approximately 30 percent of the total number of cracks were of this type.

Usually longitudinal cracking developed on only one side of the longitudinal joint. In a short section, however, between stations 582 and 704 longitudinal cracking had occurred on both sides of the longitudinal joint in 23 panels. This is the section containing the greatest amount of longitudinal cracking and the cracks are 2 feet or more from the center joint. Figure 5 shows typical longitudinal cracks in this pavement in 1943 while figure 6 shows the condition of some of the cracks observed in 1944.

The traffic was approximately the same over the full length of the section and an examination of the subgrade data, table 4, indicates that there was no important difference in the subgrade between that part of the pavement where there was considerable longitudinal cracking and the part which had a negligible amount. It is possible, however, that there may have been some

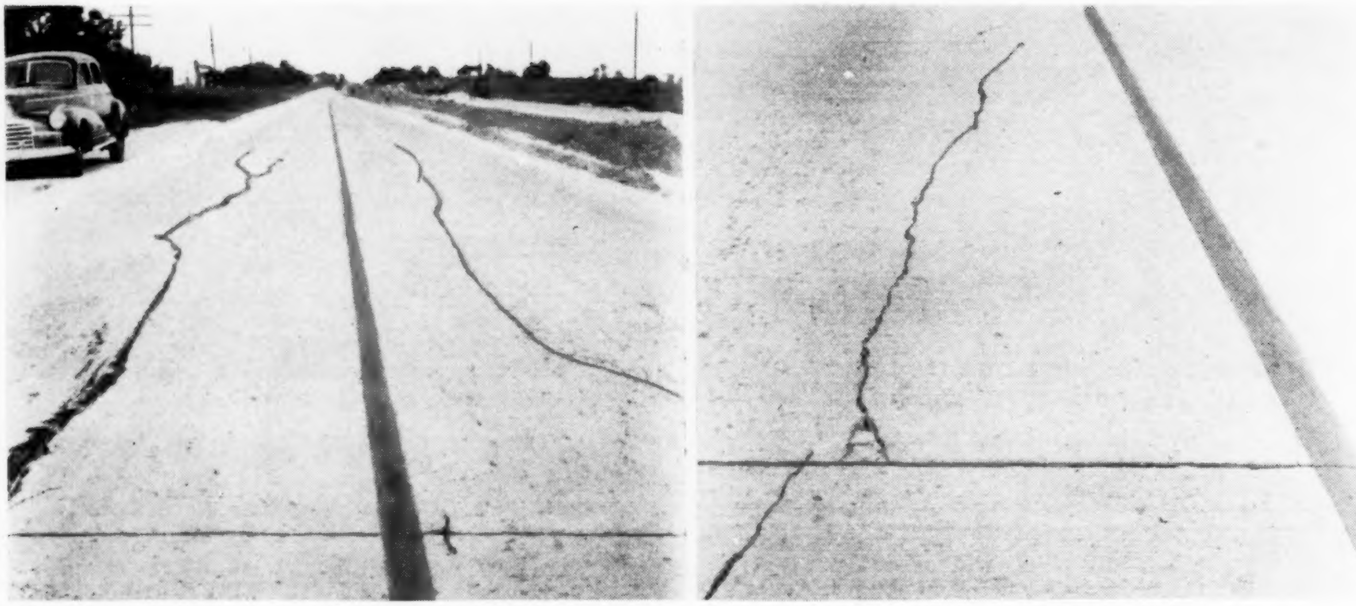

FIGURE 5.—TYPICAL LONGITUDINAL CRACKS OBSERVED IN 1943 IN INDIANA PAVEMENT SN–FA 13–B (2).

FIGURE 6.—TYPICAL LONGITUDINAL CRACKS OBSERVED IN 1944 IN INDIANA PAVEMENT SN–FA 13–B (2).

variation in the density or moisture condition of the subgrade.

The fact that the longitudinal cracking developed mainly in those parts of the pavement laid late in the construction season may suggest that this part of the pavement was opened to traffic at an earlier age than the other parts. The investigators were informed that this was not true as the different parts of the pavement were opened to traffic as construction progressed.

Part of this pavement was laid during the summer and fall of 1943, construction being completed the first week in November. The cross section of this part of the pavement was 9–8–9 inches, but the depth of the ribbon in the center joint was proportionately the same as that in the 1942 pavement or one third the center depth of the pavement. In the 1944 survey of this part of the pavement no off-center cracking was observed and it appeared that the center joint had opened throughout the full length.

It is difficult to explain the difference in the performance of the part of the pavement laid in 1943 as compared to that laid in 1942. Longitudinal cracking was found in 9–8–9-inch thickened edge and 10-inch uniform thickness pavements in Kentucky and Illinois which indicates that the increase in thickness of the 1943 Indiana pavement, as compared to the 1942 pavement, probably was not responsible for the elimination of the longitudinal cracking. It was pointed out by one of the engineers of the State Highway Commission that the shoulders on the 1942 pavement were in poor condition during the winter of 1942–43 and that the shoulders and subgrade near the edges of the pavement were very wet. The fall of 1943 was comparatively dry and the shoulders were smoothed down so that drainage away from the pavement was obtained soon after construction was completed. It was also pointed out that the 1942 pavement was used by two-way traffic during the winter of 1942–43 while the 1943 pavement was used only by one-way traffic after being completed.

Indiana pavement SN–FA 74–A(2) A(3) B(5).—The longitudinal cracking in this pavement is not serious. In the 1943 survey it was found that 3 percent of the panels contained some form of longitudinal cracking while at the time of the 1944 survey the value had increased to 4 percent. More than 90 percent of the cracks were part length ranging from 2 to 8 feet in length and, as a rule, they were not more than 6 to 12 inches from the center joint.

The percentage of longitudinal cracks was greater in the parts of this pavement laid during September and October than in the parts laid earlier. All of the cracks were in panels or parts of panels in which the center joint had not fractured and at the time of the last survey longitudinal cracking had occurred in all of the panels where the center joint had not fractured. It may be expected, therefore, that little or no additional longitudinal cracking will occur in this pavement. The majority of the cracks were within the limits of the tie bars and were being held closed. Figure 7 shows typical longitudinal cracks in this pavement. From visual inspection the cracks appeared to

FIGURE 7.—TYPICAL LONGITUDINAL CRACKS IN INDIANA PAVEMENT SN–FA 74 A (2), A (3), B (5).

FIGURE 8.—LONGITUDINAL CRACKING IN THE OLDER REINFORCED PAVEMENTS WITH DUMMY CENTER JOINT LAID IN INDIANA IN 1937 (74–E).

be in approximately the same condition at the time of the 1944 survey as they were in 1943.

Indiana pavement SN–FA 74–D (5), H (2).—Only 2 full-length and 38 part-length cracks were found in this pavement. The center joint was open in all the panels that were free from longitudinal cracking so that there was good reason to expect that no additional longitudinal cracking would develop. The form of the cracking in this pavement was very similar to that described on the previous project. This pavement was constructed during warm summer weather, construction being completed early in August and this may explain why there was less longitudinal cracking than on the two surfaces previously discussed.

LONGITUDINAL CRACKING IN OLDER PAVEMENTS STUDIED

Referring to table 4 it appears that there was no difference between the subgrade on those parts of Indiana pavements SN–FA 74–A (2) A (3) B (5) and 74–D (5) H (2) that showed longitudinal cracking and in those which had no cracking. Also the subgrade on these two projects is of the same type as that on Indiana pavement SN–FA 13–B (2) where serious longitudinal cracking developed.

The question naturally arises as to why the dummy center joint caused difficulty in the 1942 pavements after it had been apparently satisfactory in Indiana and elsewhere. Older pavements with this type of joint were examined in an effort to answer this question. Two pavements, Indiana FAP 74 laid in 1939 and FAP 74–E laid in 1937, were selected for this study. The cross section of these two pavements is 9–7–9 inches, both were reinforced with welded fabric and both had been subjected to heavy truck traffic.

It was found that a small number of longitudinal cracks have formed in these pavements. The majority of these break from the center joint at some intermediate point and continue the remaining length of the panel and, as found in the 1942 pavements, the center joint had not fractured in the panels or parts of panels where the longitudinal cracking developed. Figure 8 shows typical longitudinal cracks found in these pavements.

Although the longitudinal cracking is not prevalent in the older pavements built with the dummy center joint, it is present in sufficient amount to indicate a definite tendency toward such cracking and there is evidence to indicate that it was caused by the failure of the center joint to fracture promptly as intended. This cracking undoubtedly would have caused more concern if the pavements had not been reinforced.

It will be recalled that while the medium army tanks had apparently caused a slight increase in the amount of longitudinal cracking in the 1942 Kentucky pavements they had not caused longitudinal cracking in the older pavements of standard design with a definite separation at the longitudinal joint. In Indiana there were three older concrete surfaces of standard design in the northwestern part of the State that had been used more than a year for testing this same class of tanks and it was decided to make a general inspection of these pavements for such additional information as they might provide on this question.

Two of these surfaces, Indiana FA 32–C and FA 17–A, are divided four-lane pavements laid between 1937 and 1939 and have a combined length of approximately 15 miles of two-lane pavement. The cross

TABLE 9.—*Summary of the crack survey data on the Illinois projects*

Projects and stations	Time pavement was laid	Panels in which longitudinal joint has not broken and no longitudinal cracking has occurred				Panels in which longitudinal cracking has occurred			
		1943		1944		1943		1944	
		No.	*Pct.*	*No.*	*Pct.*	*No.*	*Pct.*	*No.*	*Pct.*
Project DA-WI 4-A (1) (Left lane):									
13+68—44+31	Middle October	28	18	25	16	1	1	2	1
44+31—82+48	Late October	62	33	47	25	7	4	8	4
82+48—103+94	Early November	8	8	8	8	9	8	9	8
Project DA-WI 4-C, 4-D (Right lane):									
16+21—57+44	Late October	102	49	1	0	10	5	10	5
57+44—71+61	Early November	70	100	0	0	0	0	0	0
Project DA-WI 4-C, 4-D (Left lane):									
49+36—101+26	Middle October	142	54	110	42	9	3	11	4
101+26—164+49	Early October	121	38	68	21	10	3	11	3
164+49—216+31	Late September	133	53	80	32	2	1	3	1
216+31—272+36	Middle September	113	41	90	32	2	1	3	1
272+36—308+82	Early September	41	23	28	15	0	0	0	0
308+82—351+12	Late August	20	10	6	3	6	3	7	3
351+12—451+04	Middle August	210	42	127	26	9	2	9	2
Project SN-A 25 (7) (2): [1]									
49+80—80+30	Late October	18	12	8	5	12	8	12	8
80+30—95+57	Early November	24	32	9	12	10	13	10	13
95+57—106+12	Middle November	7	13	1	2	6	11	6	11

[1] This pavement was placed over an old pavement between stations 49+80 and 95+55.

section of each pavement is 9–7–9 inches, both are reinforced with welded fabric, and have the dummy longitudinal joint.

A total of 12 full- or part-length longitudinal cracks were found in the two pavements. These cracks had the appearance of being old, indicating that they did not occur while the tanks were using the pavement. They were all near the longitudinal joint and were being held closed by the tie bars and reinforcement.

It was observed that the heavy traffic, including army tanks, was causing pumping at many of the expansion joints of these pavements. At a number of joints transverse cracks had occurred 6 to 8 feet from the joint and the intervening short slab was depressed; in some cases sufficiently to require patching.

The third pavement, Indiana NRHM 69–H, laid in 1935, consisted of two 11-foot lanes separated by an older pavement. The outer lanes were of 9–7–9-inch cross section reinforced with a light wire fabric. There were no longitudinal cracks in this pavement. The expansion joints were spaced at intervals of 80 feet with one intermediate contraction joint. Open transverse cracks, at which the steel was apparently broken, were present in a large number of these panels and bad faulting was noted at many of these cracks. Appreciable faulting had not occurred at the transverse joints, however, apparently due to the presence of dowels in the joints.

The conclusions drawn from the investigation of the Indiana pavements are:

1. There is a delay in the fracturing of the dummy center joint sometimes for as much as a year or more. This appears to be especially true of pavements laid late in the year.
2. The longitudinal cracks have formed in parts of the pavement where there was a delay in the fracturing or actual functioning of the center joint.
3. The heavy war traffic, including medium army tanks, has not caused longitudinal cracking in the older pavements of standard design with the dummy center joint. The dummy joint was functioning in these pavements at the time of the survey.

CRACK SURVEY DATA IN ILLINOIS

Longitudinal cracking has occurred in the surfaces of four 1942 projects in this State. In the majority of the surfaces, the cracking was first noted approximately 3 months after they were opened to traffic.

The summary of the crack survey data on the pavements investigated in Illinois is presented in table 9. An extra column has been added to the table to show the number of panels in which the center joint had not fractured and no longitudinal cracking had developed. The number of such panels was much greater in the Illinois pavements than in those of Kentucky and Indiana.

Illinois pavement DA–WI 4–A(1).—At the time of the 1943 survey it was found that 22 percent of the panels had neither fractures of the center joint nor longitudinal cracking. In the 1944 survey the percentage had decreased to 18. Longitudinal cracking was found in slightly less than 4 percent of the panels at the time of the 1943 survey and slightly more than 4 percent at the time of the 1944 survey. Approximately 30 percent of the cracks were full length.

Illinois pavement DA–WI 4–C, 4–D.—The number of panels in this surface with neither fracture of the center joint nor longitudinal cracking amounted to 42 percent in the 1943 and 23 percent in the 1944 survey. Longitudinal cracking was found in approximately 2 percent of the panels in 1943 and 2½ percent in 1944. Approximately 14 percent of the cracks were full length.

Illinois pavement SN–A–FA 25(7) (2).—The part of this pavement between stations 49+80 and 95+57 is resurfacing placed on an old concrete pavement while the remainder was laid on the natural subgrade. In the 1943 survey it was found that there was longitudinal cracking in 10 percent of the panels in the resurfacing and in 11 percent of those laid on the natural subgrade. Approximately 35 percent of the cracks were full length and there was no appreciable increase in the number of cracks in 1944 compared to 1943.

The number of panels, over the full length of the section studied, in which the center joint had not fractured and in which there was no longitudinal cracking, amounted to 17 percent in the 1943 survey and 6 percent in the 1944 survey.

Illinois pavement FA 133–D.—One crack survey was made on this surface during the fall of 1943. Only a small number of longitudinal cracks were found. At the age of 1 year of the 605 panels examined, the center joint had not fractured in 95 or 15 percent of the panels.

As in the other States, cores were drilled at the longitudinal joint in a number of panels to disclose the condition of the joint, especially in those panels where surface inspection indicated that the joint had not fractured. Figure 9 shows several typical cores.

THICKNESS OF ILLINOIS PAVEMENTS A FACTOR IN LONGITUDINAL CRACKING

Core A of figure 9 was taken from a panel with a full-length longitudinal crack approximately 2 feet from the center joint. The top edge of the ribbon was one-half inch below the surface and there was no crack either above or below the ribbon. There was no crack in the

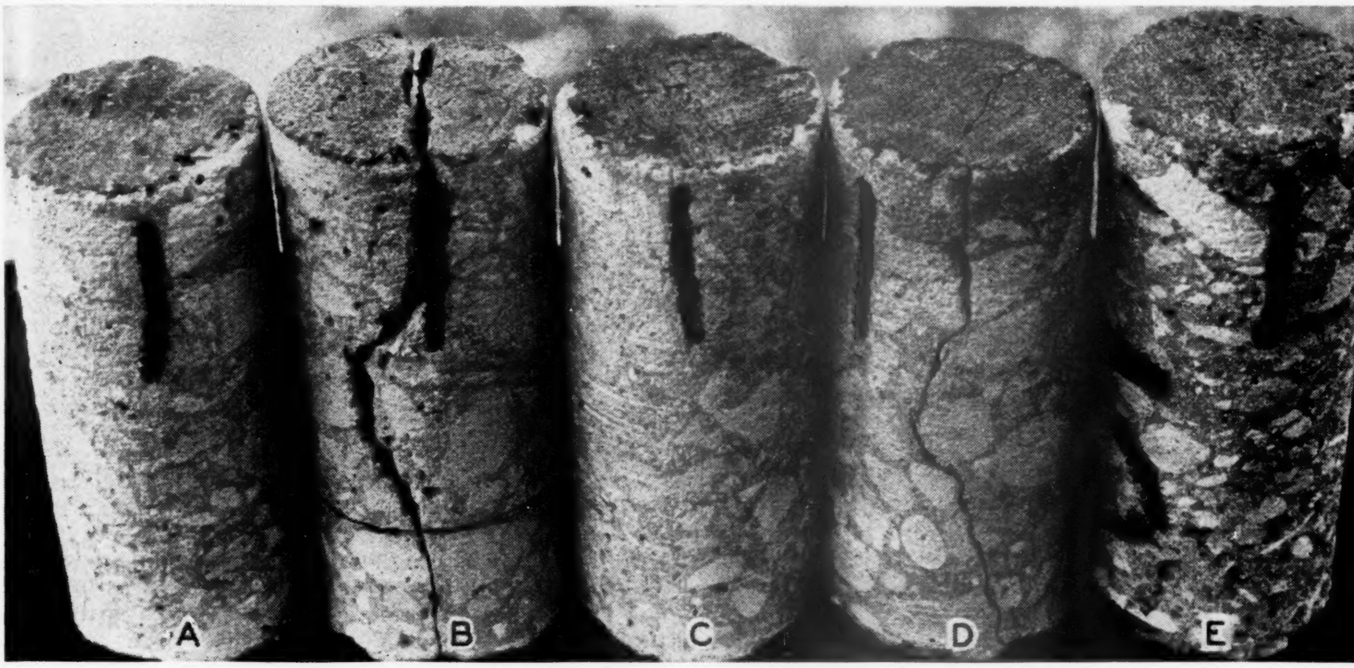

FIGURE 9.—TYPICAL CORES DRILLED AT THE LONGITUDINAL JOINT IN THE 1942 ILLINOIS PAVEMENTS.

panel from which core B was taken. The ribbon was seven-sixteenth inch below the surface and the center joint had fractured. The panel from which core C was taken showed a full-length longitudinal crack. The ribbon was one-fourth inch below the surface and there was no crack either above or below the ribbon. The panel from which core D was taken was cracked. The ribbon was one-half inch below the surface and there was no crack either above or below the ribbon. The panel from which core E was taken contained a full-length longitudinal crack. The ribbon was one-half inch below the surface and there was no crack above or below the ribbon.

As seen from the above data the ribbon forming the center joint was frequently found one-half inch below the surface. Out of 19 cores taken at the center joint the ribbon was within one-fourth inch or less of the surface in 5 cores. The center joint had not fractured, at the time of the 1943 survey, in three of the panels from which these five cores were taken and longitudinal cracking had developed in two of the three panels.

Many of the cracks in the Illinois pavements have formed fairly close to the longitudinal joint and these are being held closed by the tie bars in the joint. In a number of cases, however, the cracks are 2 feet or more from the center joint and it is expected that these cracks will open and perhaps seriously affect the condition of the pavements. Figure 10 shows several typical cracks in the Illinois pavements. The pattern of cracking was approximately the same on the resurfacing as on the other parts of the pavement. With one exception the longitudinal cracks are in panels in which the center joint had not fractured. There were no cases of longitudinal cracking on both sides of the longitudinal joint.

In studying the data from the pavements in Illinois it was noted that there were a large number of panels in which the center joint had not fractured and in which there was no longitudinal cracking. Only a small number of such panels were found in the pavements investigated in Kentucky and Indiana. This difference is attributed to the greater thickness of the Illinois pavements. The cross section of the Illinois pavements in which longitudinal cracking has developed is 10-inch uniform thickness while in Kentucky and Indiana the cross sections are 9–8–9 and 9–7–9 inches respectively.

A study of the data from the Illinois pavements indicates that in those parts of the pavement in which the center joint had not fractured and there has been no longitudinal cracking the tendency has been for the center joint to fracture rather than for longitudinal cracking to occur. This is shown by the fact that during the interval between the two surveys there was only a small increase in the number of longitudinal cracks while, there was a marked decrease in the number of panels with an uncracked center joint and no longitudinal cracking.

As was the case with the pavements investigated in Kentucky and Indiana the amount of longitudinal cracking in the Illinois pavements varies with the time of the year at which the pavement was laid. In the three surfaces included in table 9 the number of panels in which longitudinal cracking was found, at the time of the 1943 survey, amounted to 2, 1, 4, and 8 percent, respectively, in the parts of the pavement laid during August, September, October, and November.

The subgrade data for the Illinois pavements, table 5, does not show any important differences in the characteristics of the subgrade for the parts of the pavement with different amounts of longitudinal cracking. In this connection it will be recalled also that on Illinois project SN–A–FA 25 (7) (2) the amount of longitudinal cracking was approximately the same on the resurfacing as on the other part of the pavement laid on the natural subgrade. The surface drainage appeared to be excellent on all the pavements studied and only a negligible amount of transverse cracking was observed.

In the older Illinois pavements of standard design,

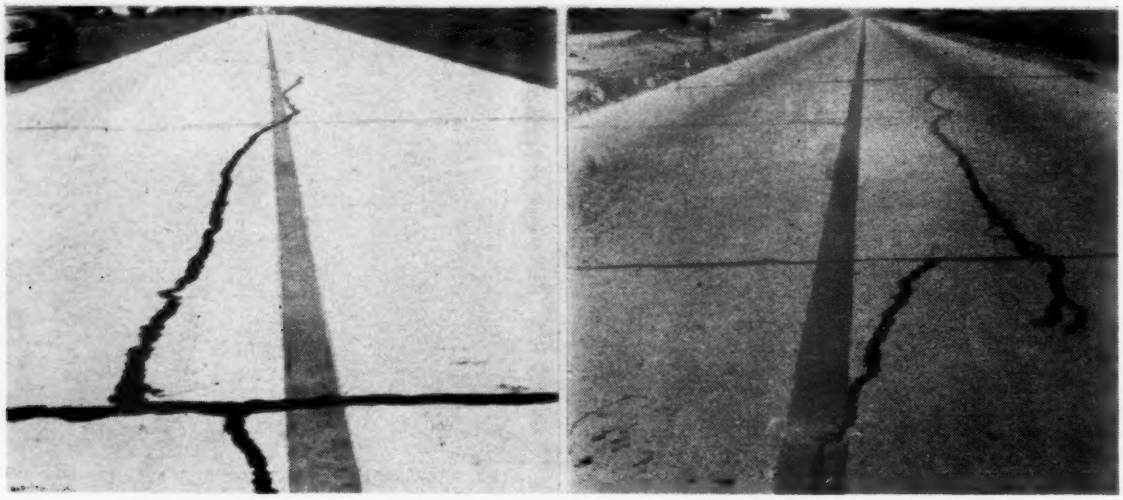

FIGURE 10.—TYPICAL LONGITUDINAL CRACKS IN THE 1942 PAVEMENTS OF ILLINOIS.

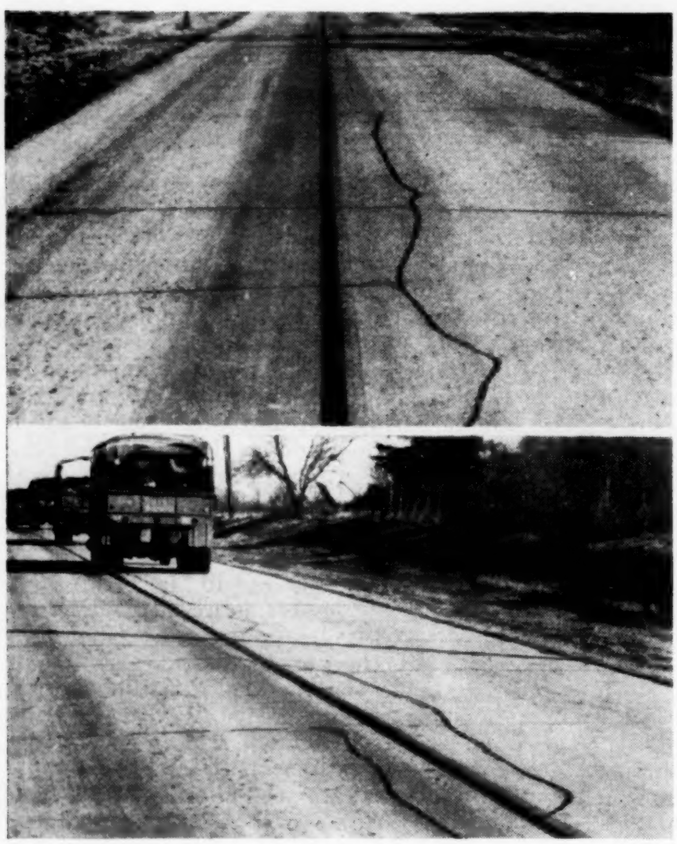

FIGURE 11.—LONGITUDINAL CRACKING IN NEBRASKA PAVEMENTS WITH DUMMY CENTER JOINT, LAID IN 1935–36.

that have been subjected to the same traffic conditions since 1942 as the 1942 pavements studied, longitudinal cracking has not developed. In these, complete partition along the center line was obtained with a deformed metal plate.

The conclusions that may be drawn from the investigation of the Illinois pavements are much the same as those indicated by the examination of the Kentucky and Indiana pavements except as affected by the greater thickness of the Illinois pavements. The delay in the fracturing of the center joint was greater in the case of the Illinois pavements and it was found in pavements laid during warm summer weather as well as in those laid late in the year. There was also a greater delay in the development of longitudinal cracks in the panels in which the center joint had not fractured.

While it was necessary to limit the scope of the investigation to Kentucky, Indiana, and Illinois, the type of longitudinal cracking found in these States has been observed in certain pavements of other States in which the dummy center joint has been used. There are probably few instances where it has been as pronounced as in the 1942 pavements described in this report but there is little doubt that a definite tendency toward this type of cracking exists with the dummy center joint. The greater amount of heavy traffic during the war may have increased this tendency, while the more limited use of reinforcement in the pavements and in some cases the elimination of tie bars in the center joints increased the seriousness and probably permitted a more rapid development of cracking. Figure 11 shows typical longitudinal cracks observed in certain pavements in Nebraska. The cross section of these pavements is 9–7–9 inches and the center joint was formed with a ⅜- by 2½-inch premolded bituminous strip.

DEFINITE SEPARATION AT CENTER JOINT AT TIME OF CONSTRUCTION FOUND NECESSARY

This survey leads to the conclusion that the important cause of the longitudinal cracking that has developed in certain pavements of Kentucky, Indiana, and Illinois is the failure of the dummy longitudinal joint to crack through promptly as intended. In the 9–8–9 and 9–7–9-inch thickened-edge pavements of Kentucky and Indiana the delay in the fracturing of the center joints of this type was found mainly in the pavements that had been laid in the fall, but in the 10-inch uniform-thickness pavements of Illinois these joints have not always ruptured promptly even when the pavement was laid in the warm summer weather.

Consideration of the various conditions that tend to produce transverse tensile stress indicates that in a pavement that does not have complete partition of the concrete along the center line, the most important cause of transverse tensile stress in the region of the center line is restrained temperature warping. That this one

cause can produce stresses sufficiently great to break the full depth slab is evidenced by the many miles of center cracking in pavements built without a center joint 25 or 30 years ago and never subjected to heavy wheel loads. The studies of warping stresses conducted by the Public Roads Administration have furnished experimental evidence to the same effect.

Heavy wheel loads also produce high tensile stress in concrete slabs of usual thicknesses. However, except for loads applied at slab corners, the highest tensile stress is found directly under the wheels. Thus, even a heavy wheel load would not produce a high transverse tensile stress at the center line of the pavement unless the wheel were over the center line. With the vehicle in its normal position in one lane the transverse stress produced at the center line of the pavement would not be large. Any stress caused by the wheel load would combine with the warping stress that happened to be present, however.

In a pavement slab that has not broken along the center line restrained temperature warping causes a transverse tensile stress that reaches its maximum value at the center line of the pavement but has a magnitude almost as great for an appreciable distance on either side of the center line. Since this region of relatively high warping stress extends out into the vicinity of normal vehicle wheel travel, an unbroken dummy joint along the center line may result in the development of very high combined stresses in that part of the pavement where the heavy wheel loads are most frequent.

Partition of the slab along the center line greatly reduces the magnitude of the temperature warping stresses for given temperature conditions. The distribution of warping stress across the pavement is also changed and for a given wheel load the combined stress is greatly reduced.

This brief discussion of slab stresses has been introduced because it is believed that it offers a plausible explanation for the conditions that have been observed during this survey.

The pavements studied in this investigation, in which londitudinal cracking has occurred, have probably all been subjected to a greater amount of heavy traffic than they would have under normal conditions. It is probable also that had they not been subjected to this abnormally heavy traffic, the longitudinal joint would have broken as intended in a larger percentage of the panels before longitudinal cracking developed in the slabs themselves. While heavy traffic probably has been a contributing cause, these same traffic conditions have not caused longitudinal cracking in the parts of the 1942 pavements in which there was a definite separation at the longitudinal joint or in the older pavements of standard design in which there was also a definite separation at the longitudinal joint. This indicates that, even with traffic conditions as they have been, longitudinal cracking would not have occurred if the center joint had cracked as intended in all of the pavements.

It is indicated that the dummy longitudinal joint, as it was designed for the 1942 pavements, was defective in that it did not always develop promptly the plane of fracture as was intended. The remedy is a matter of design.

Past experience with the dummy longitudinal joint in highway pavements has generally been with slabs having a center depth of 8 inches or less. The usual practice has been to provide a groove 2 to 2½ inches in depth to create the weakened plane. The depth of the groove has been from one-fourth to one-third of the slab depth.

The 1942 pavements in Kentucky, Indiana, and Illinois were provided with a ribbon separation that extended nearly one-third of the pavement depth. The survey shows that these joints did not always fracture as was intended, which leads to the conclusion that a groove of this depth is not adequate.

The most logical remedy for longitudinal cracking of the type studied is the use of a longitudinal joint that will have definite separation at the time of construction. If the weakened-plane center joint is to be used it should be modified in some manner to give prompt fracture. Increasing the depth of the surface groove might be helpful. Were it not for the tie bars in the joint this groove could be carried to any desired depth. Since it is not desirable to have the bottom of the groove too close to the tie bars it may be necessary to place the tie bars at a slightly greater depth below the surface than is customary.

BEHAVIOR OF ASPHALTS IN THIN-FILM OVEN TEST

BY THE DIVISION OF PHYSICAL RESEARCH, PUBLIC ROADS ADMINISTRATION

Reported by R. H. LEWIS, Senior Chemist, and W. J. HALSTEAD, Associate Chemist

IN APRIL 1941, a report was published by Lewis and Welborn on "The Properties of the Residues of 50–60 and 85–100 Penetration Asphalts from Oven Tests and Exposure." [1] This report presented the results of tests made on the residues from the standard oven test and from the thin-film oven test. The original asphalts were of the 50–60 and 85–100 grades. The thin-film oven test is the same as the standard oven test except that a 50-milliliter sample of asphalt is exposed in a flat-bottomed container 5.5 inches in diameter, the depth of the sample being approximately one-eighth inch.

The residues from the standard oven test were shown to be only slightly altered, and it was concluded that this test did not furnish adequate information concerning the probable behavior of asphalts for use in hot-mix paving. On the other hand, tests of the residues from the thin-film oven test showed that the asphalts were greatly altered by the test and, further, that the alterations were similar to those that have occurred in bitumens recovered from both laboratory and commercial paving-plant mixtures. It was concluded that this test would be useful in predicting the behavior of asphalts under processing and service conditions.

It was indicated that if an asphalt of the 50–60 grade produced a residue in the thin-film oven test (⅛-inch film, 5 hours heating at 325° F.) having 50 percent of the original penetration at 77° F. and a ductility over 40 centimeters at 77° F., there would be little danger of undue alteration occurring in the hot-mixing process and in service. To insure satisfactory performance by asphalts of the 85–100 penetration grade, requirements that the residue should have not less than 50 percent of the original penetration at 77° F. and a ductility of more than 100 centimeters at 77° F. were suggested.

The purpose of this study was to obtain similar information on representative samples of the 60–70, 100–120, and 120–150 penetration grades.

[1] PUBLIC ROADS, vol. 22, No. 2, April 1941. Also Proceedings of Association of Asphalt Paving Technologists, vol. 12, Dec. 1940.

STORED SAMPLES SHOWED CONSIDERABLE SURFACE HARDENING

The samples used for this study were from the same producers, and had been submitted at the same time as those tested and reported by Lewis and Welborn [1] in 1941 and in their earlier report on "The Physical and Chemical Properties of Petroleum Asphalts of the 50–60 and 85–100 Penetration Grades." [2] Samples were selected so as to include at least one sample from each general source of base petroleum, and also one sample of each asphalt from a source or producer that had supplied a sample of the 50–60 or 85–100 grade showing unusual behavior in the tests.

Seventeen asphalts from different sources or produced by different methods of manufacture were chosen. Data on the source and method of refining are shown in table 1. To facilitate comparisons, samples from the same source as samples used in the previous reports carry the same identification numbers. Samples corresponding to samples 20 and 29 of the 60–70 grade and samples 6 and 13 of the 120–150 grade were not available. A sample of the 40–50 grade of sample 29 was tested and the results listed with those for the 60–70 grade.

The samples had been stored in unopened metal containers for approximately 6 years and most of them gave evidence of surface hardening. It was believed that the surface material was oxidized or otherwise altered. To obtain a sample having more nearly the original characteristics, the top one-quarter inch of asphalt was removed from each can. The balance of the material was prepared for test by placing the container in an oven and heating at 230° F. until the asphalt was liquid enough to stir easily. The material was then stirred thoroughly.

Since the amount of hardening was of interest, the material cut from the top was tested for penetration at

[2] PUBLIC ROADS, March 1940, vol. 21, No. 1. Also Proceedings of Association of Asphalt Paving Technologist, vol. 11, January 1940.

TABLE 1.—*Source and method of refining asphalt cement*

Identification No.	Producer identification	Source of base petroleum	Method of refining
3	3	California, San Joaquin Valley field	Reduction and steam distillation.
6	6–A	Colombia	Vacuum distillation with pipe still.
8	8	Mexico	
9	9	Mexico, Panuco field	Steam distillation in Trumble (pipe) still.
13	12	Venezuela	Fire and steam distillation.
14	13–A	Venezuela, Mene Grande field	Continuous distillation under subatmospheric pressure with steam.
15	13–B	do	Distilled in batch stills at atmospheric pressure with steam.
19	15	Arkansas, Smackover field	Vacuum distillation at a low temperature.
20	16–A	do	Vacuum distillation, 89 m. p. flux.
29	21	Oklahoma, Healdton and Graham	
30	22	Kentucky and Illinois	Fire and steam distillation, possibly blown.
32	24–A	Kansas	Straight run, steam refined, vacuum process.
33	24–B	do	Produced from Winkler-Koch Shell still.
34	25–A	Wyoming	Fire and steam distillation.
35	25–B	Unknown	do
37	26–A	Mexico and Domestic Gulf Coast	
40	28	Kentucky	Dubbs cracking process.

TABLE 2.—*The surface hardening of asphalts in cans after aging approximately 6 years*

Identification No.	Type of metal container	Penetration at 77° F., 100 gm., 5 sec.								
		60-70 grade			100-120 grade			120-150 grade		
		Top [1]	Total [2]	Relation top to total	Top	Total	Relation top to total	Top	Total	Relation top to total
				Percent			*Percent*			*Percent*
3	(3)	34	67	51	59	103	57	100	138	72
6	Slip top, cylindrical	33	66	50	59	116	51			
8	Double friction top, cylindrical	32	65	49	53	107	50	60	130	46
9	Single friction top, cylindrical	80	80	100	102	108	94	131	132	99
13	Double friction top, cylindrical	38	60	63	55	109	50			
14	do	27	65	41	85	117	73	131	136	96
15	do	29	61	47	50	105	48	111	121	90
19	Small screw cap, rectangular	60	63	95	92	101	91	127	126	101
20	Single friction top, cylindrical				82	99	83	117	129	91
29	Small screw cap, rectangular	[4] 20	[4] 45	[4] 44	107	108	99	189	164	115
30	Single friction top, cylindrical	35	65	54	79	107	74	121	131	92
32	Clamped top, bucket type	32	62	52	53	105	50	63	121	52
33	do	32	57	56	49	97	51	54	110	49
34	Large screw cap, rectangular	34	66	52	62	102	61	88	123	72
35	Double friction top, cylindrical	37	60	62	101	117	86	115	135	85
37	Clamped top, bucket type	40	61	66	65	115	57	74	120	62
40	Small screw cap, rectangular	30	64	47	52	106	49	70	132	53

[1] Material cut from approximately the top one-fourth inch of sample.
[2] Material for test taken after removal of top one-fourth inch and sample had been heated and stirred thoroughly.
[3] 60-70 material in large screw-cap, rectangular container; 100-120 material in single friction top, cylindrical container; 120-150 material in small screw-cap, rectangular container.
[4] These values are for 40-50 grade.

TABLE 3.—*Test characteristics of the 60-70 penetration grade asphalts and their residues from the thin-film oven test*

Identification No.	Original asphalt									Thin-film oven test (⅛-in. film, 5 hr. at 325° F.)							
	Oliensis spot test	Specific gravity at 77° F.	Softening point	Penetration, 100 gm. 5 sec. at—			Ductility, 5 cm. per min. at—			Change in weight	Tests on the residue						
											Softening point	Penetration, 100 gm. 5 sec. at—			Ductility, 5 cm. per min. at—		
				50° F.	60° F.	77° F.	50° F.	60° F.	77° F.			50° F.	60° F.	77° F.	50° F.	60° F.	77° F.
			°F.				*Cm.*	*Cm.*	*Cm.*	*Percent*	*°F.*				*Cm.*	*Cm.*	*Cm.*
3	Negative	1.013	119	9	19	67	195	250+	250+	−0.08	126	6	13	41	[1] 0	250+	250+
6	do	1.006	126	14	24	66	8	43	219	+.06	139	11	18	41	4.5	9	114
8	Positive	1.041	131	20	30	65	5.5	11.5	42	−.26	148	13	19	36	3	4	7.5
9	Negative	1.039	126	20	33	80	32	201	250+	−.65	139	14	22	46	6	16.5	108
13	Positive	1.025	132	17	28	60	6.5	15	146	−.19	151	12	17	32	3.5	4.5	9
14	do [2]	1.024	124	13	23	65	8.5	142	250+	+.17	133	10	17	42	5.5	12	206
15	do	1.034	127	13	22	61	9	41	250+	−.004	141	10	15	35	4	6.5	45
19	Negative	1.024	124	13	23	63	6.5	57	236	+.11	133	10	18	43	5.5	10	187
29 [3]	Positive	1.039	126	9	15	45	4.5	17	250+	−.07	144	6	11	24	[1] .5	3.5	10
30	Negative	1.005	127	15	25	65	7.5	23.5	193	+.08	140	12	20	44	4.5	7	38
32	do	1.012	129	13	24	62	6	25	166	+.11	140	10	17	40	4.5	7	34
33	Positive	1.034	128	14	23	57	4.3	8.5	36	+.05	151	11	19	39	3	4	6
34	do	1.037	128	18	29	66	6.5	16.5	180	+.07	146	14	20	36	3.5	4.5	12
35	Negative	1.011	131	18	27	60	5.3	11.5	167	−.09	139	14	20	40	4	6	22
37	do	1.011	130	19	29	61	8	24	144	−.29	146	14	20	40	4.5	6	31
40	Positive	1.052	123	13	22	64	8.8	57	250+	−2.10	148	7	10	22	[1] 0	3	8

[1] Specimen broke. [2] Very slight spot. [3] 40-50 grade; no 60-70 grade available.

77° F. (100 grams, 5 seconds). Comparison of the penetration of this top material with that of the stirred sample, which will be referred to as the total sample, is shown in table 2. This table also shows the type of metal container in which each material had been stored.

A majority of the samples showed considerable hardening at the surface. The exceptions were all grades of samples 9 and 19; the 100-120 and 120-150 grades of sample 29, and the 120-150 grades of samples 14, 15, 20, and 30. In all these samples the penetration of the top material was equal to 90 percent or more of the penetration of the total sample. Sample 29 of the 120-150 grade was unusual in that the top material was softer than the total sample. When opened, the surface of this sample had drops of water on it, even though the material was in a container with a small, well-sealed opening. Considerable difficulty was experienced from foaming when heating this material.

There was no definite trend in the amount of hardening. For example, in the case of sample 14, the top material of the 60-70 grade had a penetration equal to 41 percent of the penetration of the total sample. For the 100-120 grade the penetration of the top material was 73 percent of that of the total sample and there was practically no difference for the 120-150 grade, the top having a penetration of 96 percent of that of the total sample. All of these samples were in similar containers and approximately the same amount of material was in each can. Sample 29, which was in cans having a small screw cap, also showed variation in penetration ratios to a marked degree. The top of the 40-50 grade had a penetration of 44 percent of that of the total sample; in the 100-120 grade it had 99 percent: and in the 120-150 grade, 115 percent.

However, despite the surface hardening of these samples there is no indication that the bulk of the material had changed appreciably since, in most cases, the penetration was still within the required limits for

TABLE 4.—*Test characteristics of the 100–120 penetration grade asphalts and their residues from the thin-film oven tests*

Identification No.	Original asphalt: Oliensis spot test	Original asphalt: Specific gravity at 77° F.	Original asphalt: Softening point	Original asphalt: Penetration, 100 gm. 5 sec. at — 50° F.	Penetration 60° F.	Penetration 77° F.	Original asphalt: Ductility, 5 cm. per min. at — 50° F.	Ductility 60° F.	Ductility 77° F.	Thin-film oven test (1/8-in. film, 5 hr. at 325° F.): Change in weight	Tests on the residue: Softening point	Tests on the residue: Penetration, 100 gm. 5 sec. at — 50° F.	Penetration 60° F.	Penetration 77° F.	Tests on the residue: Ductility, 5 cm. per min. at — 50° F.	Ductility 60° F.	Ductility 77° F.
			°F.				Cm.	Cm.	Cm.	Percent	°F.				Cm.	Cm.	Cm.
3	Negative	1.011	114	14	29	103	250+	250+	175	−0.20	119	9	17	61	62	250+	250+
6	do	1.002	113	22	41	116	185	250+	152	+.03	122	18	29	70	11.5	84	185
8	Positive	1.036	119	30	47	107	12.5	45	98	−.50	144	15	23	47	3.5	5	13
9	Negative	1.036	120	27	46	108	105	250+	221	−.88	135	15	24	56	7.5	20	115
13	Positive	1.027	119	29	47	109	36	88	215	−.33	137	17	25	49	5.5	8	31
14	do [1]	1.018	115	23	41	117	64	250+	225	+.11	123	15	26	67	10.5	62	180
15	do	1.027	117	23	39	105	66	250+	152	+.01	128	14	23	53	5.5	21	194
19	Negative	1.019	115	21	37	101	23	250+	144	+.12	125	16	27	65	8	43	230
20	do	1.013	121	32	49	99	11.5	46	165	−.77	135	22	32	62	5	8	60
29	Positive	1.026	114	16	33	108	66	250+	140	−.04	127	10	18	46	5.5	13	250+
30	Negative	.982	119	30	46	107	15	66	106	+.07	126	24	37	80	8	14.5	98
32	do	1.007	116	21	37	105	15.5	140	158	+.15	129	17	27	62	6.5	15.5	149
33	Positive	1.028	119	20	35	97	10	14	85	+.01	138	16	25	57	5	6	9.5
34	do	1.031	117	24	40	102	31	130	160	−.17	131	15	23	51	4.5	10	60
35	Negative	1.006	112	26	45	117	25	125	124	−.17	121	20	33	78	9	19.5	160
37	do	1.010	118	33	52	115	17	144	166	−1.60	139	18	27	53	4.5	7.5	35
40	Positive	1.049	111	20	37	106	57	250+	165	−2.38	140	8	13	29	[2] 1.5	4.8	18

[1] Very slight spot. [2] Specimen broke.

TABLE 5.—*Test characteristics of the 120–150 penetration grade asphalts and their residues from the thin-film oven test*

Identification No.	Original asphalt: Oliensis spot test	Original asphalt: Specific gravity at 77° F.	Original asphalt: Softening point	Original asphalt: Penetration, 100 gm. 5 sec. at — 50° F.	Penetration 60° F.	Penetration 77° F.	Original asphalt: Ducility, 5 cm. per min. at — 50° F.	Ductility 60° F.	Ductility 77° F.	Thin-film oven test (1/8-in. film, 5 hr. at 325° F.): Change in weight	Tests on the residue: Softening point	Tests on the residue: Penetration, 100 gm. 5 sec. at — 50° F.	Penetration 60° F.	Penetration 77° F.	Tests on the residue: Ductility, 5 cm. per min. at — 50° F.	Ductility 60° F.	Ductility 77° F.
			°F.				Cm.	Cm.	Cm.	Percent	°F.				Cm.	Cm.	Cm.
3	Negative	1.008	108	19	41	138	250+	250+	142	−0.34	114	14	26	84	250+	250+	147
8	Positive	1.033	114	31	53	130	30	113	92	−.51	138	20	29	57	5	7.5	23
9	Negative	1.033	113	33	55	132	183	225	161	−.78	132	20	33	70	12	47.5	88
14	Positive [1]	1.016	109	25	48	136	240	250+	161	+.11	121	18	32	79	15	104	167
15	do	1.026	111	24	46	121	134	250+	132	−.04	126	17	28	62	10	31.5	167
19	Negative	1.017	111	24	44	126	62	229	110	+.11	119	18	33	80	1.5	88	139
20	do	1.012	112	35	58	129	34	155	123	−.81	126	24	38	77	7	19.5	95
29	Positive	1.022	105	23	46	164	190+	230	120	−.52	124	15	26	65	7	22.5	196
30	Negative	.983	114	32	54	131	32	133	116	+.04	121	27	45	95	11.5	42	88
32	do	1.005	112	24	43	121	68	168	130	+.12	123	20	30	72	10	28.5	93
33	Positive	1.029	112	23	42	110	14	33	81	−.10	134	18	30	65	4.5	6	10.5
34	do [1]	1.023	112	24	43	123	107	195	107	+.13	124	18	31	72	14	115	145
35	Negative	1.005	110	31	53	135	75	250+	115	−.27	118	23	38	88	12	68	157
37	do	1.004	118	40	62	120	15	32	79	−.72	140	29	39	67	5	6.5	15
40	Positive	1.048	107	25	47	132	250+	250+	105	−2.44	136	11	18	34	3.5	6.5	32

[1] Very slight spot.

the designated grades. The exceptions are sample 9 of the 60–70 grade, which had a penetration of 80; sample 20 of the 100–120 grade, which had a penetration or 99; sample 29 of the 120–150 grade, which had a penetration of 164; and all grades of sample 33, their values being 57 for the 60–70 grade, 97 for the 100–120 grade, and 110 for the 120–150 penetration grade. The results for these particular samples have been included under the original grade designations since in most cases the deviations from the required penetration were small.

The previous reports have indicated that the changes in the physical properties of asphalts as measured by the consistency and ductility are of a greater magnitude and thus of more significance than chemical changes as exemplified by solubility tests. It has also been shown that the standard oven-loss test is of little value in predicting the durability of the asphalt. Therefore, in this study only the Oliensis spot test, specific gravity at 77° F., softening point, penetration (100 grams, 5 seconds) at 50°, 60°, and 77° F., and ductility (5 centimeters per minute) at the same three temperatures were determined for the original asphalt. The thin-film oven test (one-eighth inch, 5 hours, at 325° F.) was made and the residue was tested for softening point, penetration at 50°, 60°, and 77° F., and ductility at these three temperatures. Table 3 shows the results of these tests for the 60–70 penetration grade, table 4 shows the results for the 100–120 grade, and table 5 shows the results for the 120–150 grade.

A summary of the changes in weight and the changes in penetration and softening point of the residues expressed as percentages of the original test values, are shown in table 6. This table also includes data for the 50–60 and 85–100 penetration grades which were taken from the earlier report by Lewis and Welborn. There seems to be little significance in the amount of change in weight of the samples, some show a slight decrease while others show an increase. Only six samples had losses in weight greater than 1 percent, these being all grades of sample 40, and the 100–120 grade of sample 37. In addition to these samples there were only one sample of the 60–70 grade, two samples of the 85–100 grade, two of the 100–120 grade, and five of the 120–150 grade which had a loss greater

TABLE 6.—*Changes in test characteristics for various penetration grades of asphalt after the thin-film oven test*

Identification No.	Penetration grade														
	50–60			60–70			85–100			100–120			120–150		
	Change in weight	Relation of test value of residue to original		Change in weight	Relation of test value of residue to original		Change in weight	Relation of test value of residue to original		Change in weight	Relation of test value of residue to original		Change in weight	Relation of test value of residue to original	
		Penetration at 77° F., 100 gm. 5 sec.	Softening point		Penetration at 77° F., 100 gm. 5 sec.	Softening point		Penetration at 77° F., 100 gm. 5 sec.	Softening point		Penetration at 77° F., 100 gm. 5 sec.	Softening point		Penetration at 77° F., 100 gm. 5 sec.	Softening point
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
3	+0.01	64	106	−0.08	61	106	−0.14	59	106	−0.20	59	104	−0.34	61	105
6	0.	67	109	+.06	62	110	+.04	63	106	+.03	60	108			
8	−.22	61	117	−.26	55	113	−.46	45	119	−.50	44	121	−.51	44	121
9	−.20	66	110	−.65	58	110	−.55	53	112	−.88	52	112	−.78	53	117
13	−.24	61	115	−.19	53	114	−.11	50	114	−.33	45	115			
14	+.09	63	107	+.17	65	107	+.05	59	113	+.11	57	107	+.11	58	111
15	0.	52	111	−.004	57	111	+.01	57	115	+.01	50	109	−.04	51	113
19	+.10	68	106	+.11	68	107	+.09	68	108	+.12	64	109	+.11	64	107
20	−.46	67	107				−.59	68	109	−.77	63	112	−.81	59	112
29[1]				−.07	53	114	−.13	43	114	−.04	43	111	−.52	40	118
30	+.05	71	105	+.08	68	110	+.08	69	111	+.07	75	106	+.04	73	106
32	+.08	67	107	+.11	64	109	+.09	62	111	+.15	59	111	+.12	60	110
33	+.05	72	120	+.05	68	118	+.06	60	117	+.01	59	116	−.10	59	120
34	+.04	55	116	+.07	54	114	+.04	52	115	−.17	50	112	+.13	59	111
35	−.10	72	105	−.09	67	106	−.14	68	107	−.17	67	108	−.27	65	107
37	−.13	60	111	−.27	66	112	−.12	58	114	−1.60	46	118	−.72	56	119
40	−2.09	38	124	−2.10	34	120	−2.17	32	123	−2.38	27	126	−2.44	26	127

[1] Values listed under the 60–70 grade are for the 40–50 grade.

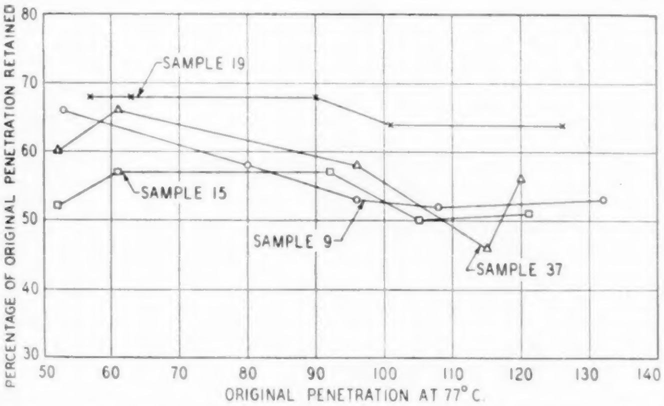

FIGURE 1.—PERCENTAGE OF PENETRATION RETAINED COMPARED TO THE ORIGINAL PENETRATION AT 77° C.

than 0.5 percent. Most specifications for the standard oven-loss test allow a maximum loss of 1 percent, and these results indicate that if the thin-film test were used in specifications, this same value would be suitable for all the grades up to and including the 120–150 penetration grade.

REQUIREMENT THAT RESIDUE HAVE 50 PERCENT OF ORIGINAL PENETRATION IS NOT UNDULY SEVERE

The percentage of original penetration retained by the residue from the thin-film oven test shows a general tendency to decrease with increases in original penetration but there are numerous deviations from this trend. In a number of cases the percentage of penetration retained by an asphalt of the 120–150 grade is greater than that retained by the corresponding asphalt of the 100–120 grade. Typical curves showing the relation between original penetration and percentage of retained penetration are shown in figure 1. As would be expected, the percentage increase in softening point of the residue, as compared with that of the original material, shows a general tendency to increase as the percentage of retained penetration decreases. However, no significant relation between increase in softening point and other properties of the materials could be established.

As previously stated, it has been suggested that, for asphalts of the 50–60 and 85–100 penetration grades, the residue from the thin-film oven test be required to have a penetration not less than 50 percent of the original penetration. Table 6 shows that such a requirement would not be unduly severe if applied to all grades between 50 and 150 penetration. The samples which fail to meet such a requirement are the 85–100, 100–120, and 120–150 grades of sample 8; the 100–120 grade of sample 13; the 85–100, 100–120, and 120–150 grades of sample 29; the 100–120 grade of sample 37, and all grades of sample 40.

WHEN COMPARED AT THE SAME PENETRATION, RESIDUE FROM THIN-FILM OVEN TEST HAS LOWER DUCTILITY THAN THE ORIGINAL ASPHALT

The ductilities of the residues from the thin-film oven test compared to those of the original asphalts at the same temperature fails to give a clear picture of the amount of change caused by the heating, since the change in the consistency may either decrease or increase the ductility depending on the original penetration.

In order to obtain a better basis for comparison, the double logarithm of the ductility was plotted against the penetration at which the ductility was obtained, the values for all grades and at all test temperatures being plotted on the same graph. It was found in most cases that the values for the asphalts, before the oven-heat test, form a smooth curve with the maximum ductility occurring at a penetration which is usually between 50 and 90 for the asphalts tested in this investigation. The data for the residues from the oven-heat test form a second curve which is below that for the original asphalts. The exceptions to this behavior are samples 8, 20, 33, 34, and 37 which have separate curves for each penetration grade. A typical curve similar to those obtained for a majority of the

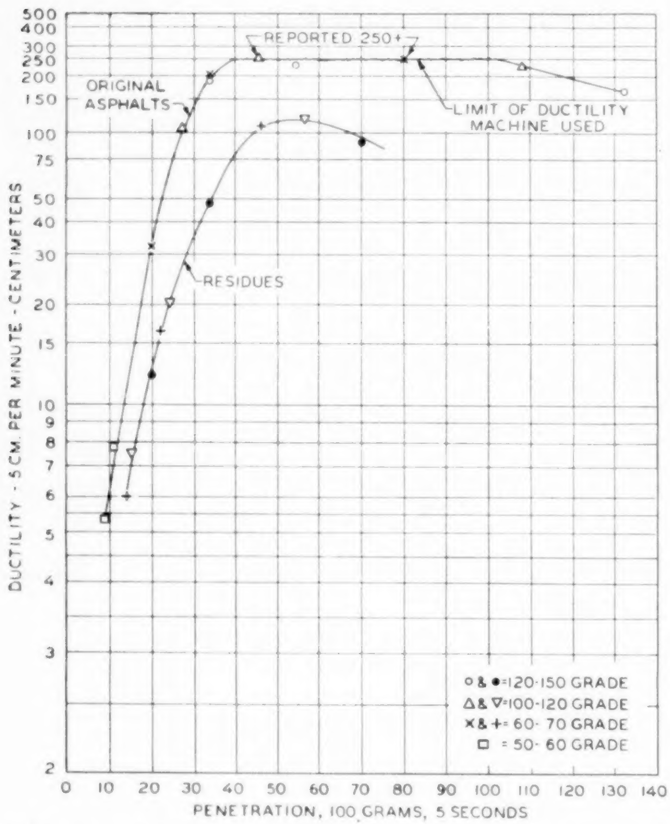

FIGURE 2.—RELATION BETWEEN DUCTILITY AND PENETRATION FOR SAMPLE 9.

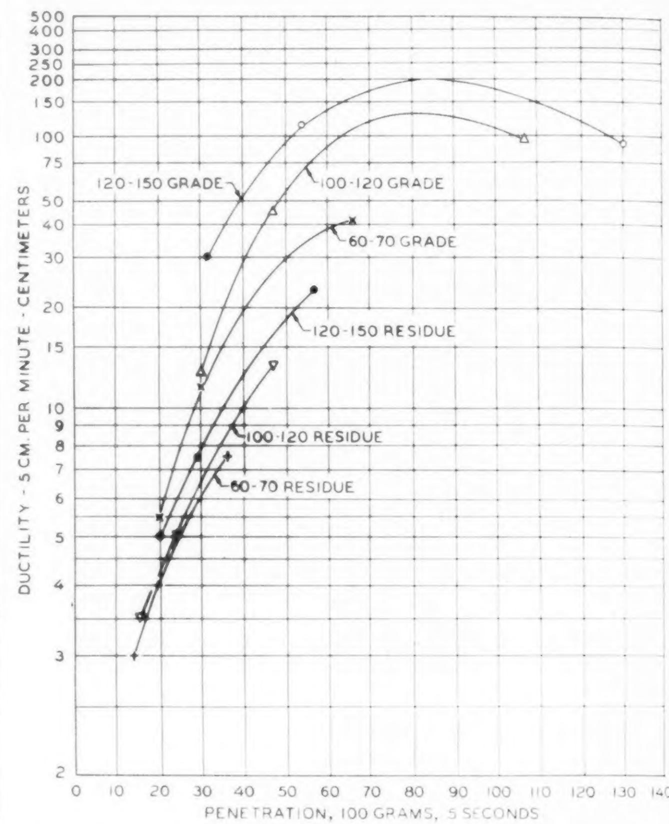

FIGURE 3.—RELATION BETWEEN DUCTILITY AND PENETRATION FOR SAMPLE 8.

samples and their residues is shown in figure 2, while figure 3 exemplifies the wide variation obtained with a few asphalts. It is interesting to note that both sample 9 of figure 2 and sample 8 of figure 3, were manufactured from Mexican petroleum, but sample 9 was steam distilled while sample 8 was refined by some process which gave a residue with a positive spot when tested by the Oliensis method.

From the curves constructed as indicated a close approximation of the ductility of each original asphalt at any penetration from approximately 10 to 130 can be made. Table 7 shows the penetration and ductility of all the residues and the ductility of the original asphalt (estimated from the curve) at the penetration of the residue. When the comparison is made on this basis it is seen that the ductility of the residue is always equal to or less than that of the original sample.

There is only a small difference between the ductilities for the lower penetrations, which approach very closely the lower limit of measurable ductility, but as the penetration becomes greater the difference increases. At 77° F., excepting the samples which have a ductility of the original material greater than 250 centimeters which makes the percentage indeterminable, only four samples have residues with a ductility equal to more than 50 percent of that of the original at the penetration of the residue, and in most cases the proportion of the ductility destroyed is much greater than this. These samples are the 100–120 grade of 20, the 100–120 grade of 32, the 120–150 grade of 34, and the 60–70 grade of 37. These results show that, in addition to the change in ductility caused by the change in consistency of the samples when they are heated in the thin-film oven test, there is an additional decrease in the ductility caused by the heating and the amount of this decrease varies considerably for the samples from various sources and methods of manufacture.

SUGGESTED REQUIREMENTS GIVE COMPARABLE RESULTS FOR ALL GRADES

Comparison of ductilities at the same penetration as shown in table 7 is impractical as a specification requirement, the actual measurement of the ductility at 77° F. being a more simple means of control. Lewis and Welborn [3] suggested ductility requirements of not less than 40 centimeters at 77° F. (5 centimeters per minute) for the 50–60 grade and not less than 100 centimeters at 77° F. for the 85–100 grade. Using these figures as the basis, the results indicate that reasonable requirements for the other grades tested would be: Not less than 40 centimeters at 77° F. (5 centimeters per minute) for the 60–70 grade and not less than 100 centimeters at 77° F. for the 100–120 and 120–150 grades. Table 8 is a summary of the asphalts of all the grades tested, indicating those that would fail to meet the proposed requirements for penetration and ductility. In general, those samples which fail in one grade fail in all grades. In all cases except the 100–120 and 120–150 grades of sample 29, the samples which fail to retain at least 50 percent of the original penetration at 77° F. also fail to meet the ductility requirement.

It has been shown in a previous report [4] that if the logarithm of the penetration is plotted against the temperature in degrees Fahrenheit a straight line will

[3] See footnote 1.
[4] See footnote 2.

TABLE 7.—*The effect of the thin-film oven test on the ductility-penetration relationship*

Identification No.	Temperature of test	Penetration grade— 60–70: Tests on residue: Penetration, 100 gm., 5 sec.	60–70: Tests on residue: Ductility, 5 cm. per min.	60–70: Approximate ductility of original asphalt at equal penetration	100–120: Tests on residue: Penetration, 100 gm., 5 sec.	100–120: Tests on residue: Ductility, 5 cm. per min.	100–120: Approximate ductility of original asphalt at equal penetration	120–150: Tests on residue: Penetration, 100 gm., 5 sec.	120–150: Tests on residue: Ductility, 5 cm. per min.	120–150: Approximate ductility of original asphalt at equal penetration
	°F.		Cm.	Cm.		Cm.	Cm.		Cm.	Cm.
3	77	41	250+	250+	61	250+	250+	84	147	250+
	60	13	250+	250+	17	250+	250+	26	250+	250+
	50	6	[1] 0	[2]	9	62	[2]	14	250+	250+
6	77	41	114	250+	70	185	250+	...	...	...
	60	18	9	28	29	84	220	...	...	...
	50	11	4.5	6	18	11.5	28	...	...	...
8	77	36	7.5	15	47	13	45	57	23	120
	60	19	4	5.5	23	5	8.5	29	7.5	24
	50	13	3	3	15	3.5	3.5	20	5	10
9	77	46	108	250+	56	115	250+	70	88	250+
	60	22	16.5	50	24	20	70	33	47.5	190
	50	14	6	10.5	15	7.5	13	20	12	32
13	77	32	9	29	49	31	100	...	...	...
	60	17	4.5	6.5	25	8	17	...	...	...
	50	12	3.5	4	17	5.5	6.5	...	...	...
14	77	42	206	250+	67	180	250+	79	107	250+
	60	17	12	22	26	62	220	32	104	250+
	50	10	5.5	5.5	15	10.5	15	18	15	36
15	77	35	45	250+	53	194	250+	62	167	250+
	60	15	6.5	12	23	21	90	28	31.5	250+
	50	10	4	5	14	5.5	9.5	17	10	15
19	77	43	187	250+	65	230	250+	80	139	250+
	60	18	10	10	27	43	120	33	88	250+
	50	10	5.5	5.5	16	8	8	18	11.5	11.5
20	77	...	...	...	62	60	80	77	95	200
	60	...	...	...	32	8	18	38	19.5	43
	50	...	...	...	22	5	5	24	7	12
29 [3]	77	24	10	250+	46	250+	250+	65	196	250+
	60	11	3.5	6.5	18	13	200	26	22.5	250+
	50	6	[1] .5	[2]	10	5.5	5.5	15	7	30
30	77	44	38	80	80	98	215	95	88	210
	60	20	7	17	37	14.5	46	45	42	85
	50	12	4.5	6	24	8	17	27	11.5	21
32	77	40	34	160	62	149	220	72	93	220
	60	17	7	7.5	27	15.5	75	30	28.5	100
	50	10	4.5	4.5	17	6.5	7.5	20	10	12
33	77	39	6	38	57	9.5	72	65	10.5	90
	60	19	4	9.5	25	6	15	30	6	23
	50	11	3	4	16	5	7	18	4.5	9
34	77	36	12	36	51	60	180	72	145	200
	60	20	4.5	7.5	23	10	31	31	115	160
	50	14	3.5	4	15	4.5	6	18	14	110
35	77	40	22	170	78	160	250+	88	157	250+
	60	20	6	6	33	19.5	95	38	68	150
	50	14	4	4	20	9	9	23	12	12
37	77	40	31	60	53	35	125	67	15	40
	60	20	6	8.5	27	7.5	20	39	6.5	15
	50	14	4.5	6.5	18	4.5	8	29	5	12
40	77	22	8	57	29	18	250+	34	15	250+
	60	10	3	5	13	4.7	8.7	18	6.5	30
	50	7	[1] 0	3	8	[1] 1.5	4	11	3.5	5.5

[1] Specimen broke.
[2] Could not be estimated.
[3] Values listed under 60–70 grade are for 40–50 penetration grade.

connect the points. The slope of this line is a measure of the susceptibility of the material to changes in consistency with temperature changes.

These slopes may be determined from the graph or calculated from any two values of the penetration and the temperatures at which those penetrations occur with the following equation:

$$\text{slope} = \frac{\log p_2 - \log p_1}{t_2 - t_1}$$

where p_2 and p_1 are the penetrations (100 grams, 5 seconds) at the two temperatures, t_2 and t_1, respectively.

Table 9 shows these slopes for the asphalts tested for the five penetration grades from 50–60 through 120–150. The slopes for the 60–70, 100–120, and 120–150 grades were calculated by the equation using the penetrations at 77° F. and at 50° F. given in tables 3, 4, and 5. The slopes for the 50–60 and 85–100 grades were taken from an earlier report.[5] A difference of one point in the pene-

[5] See footnote 2.

TABLE 8.—*Identification of the asphalts which fail to meet the proposed requirements for the thin-film oven test* [1]

Identification No.	50–60 penetration		60–70 penetration		85–100 penetration		100–120 penetration		120–150 penetration	
	Proposed requirement									
	Penetration at 77° F. 50+ percent	Ductility at 77° F. 40+ cm.	Penetration at 77° F. 50+ percent	Ductility at 77° F. 40+ cm.	Penetration at 77° F. 50+ percent	Ductility at 77° F. 100+ cm.	Penetration at 70° F. 50+ percent	Ductility at 77° F. 100+ cm.	Penetration at 77° F. 50+ percent	Ductility at 77° F. 100+ cm.
3										
6									N	N
8		O		O	X	X	X	O	X	O
9										X
13		X		X		X	X	X	N	N
14										
15		X								
19										
20		O	N	N		X		X		X
29 [2]	N	N		X	X	X	X		X	
30		X		X				X		X
32		X		X		X				X
33		O		O		X		O		O
34		X		X		X		X		
35				X						
37		X		X		X	X	X		O
40	X	X	X	X	X	X	X	X	X	X

[1] Symbols used are as follows:
X denotes the samples which fail to meet the requirement; O denotes those samples which fail to meet the requirement and also had an initial ductility of less than 100 cm. at 77° F.; and N denotes no test.
[2] Data listed under 60–70 grade are for 40–50 grade.

TABLE 9.—*The effect of the thin-film oven test on the susceptibility to changes in consistency with changes in temperature*

Identification No.	Slope of log penetration-temperature curve for—							
	50–60 grade original	60–70 grade		85–100 grade original	100–120 grade		120–150 grade	
		Original	Residue		Original	Residue	Original	Residue
3	30×10^{-3}	32×10^{-3}	31×10^{-3}	32×10^{-3}	32×10^{-3}	31×10^{-3}	32×10^{-3}	29×10^{-3}
6	25	25	21	27	27	22		
8	19	19	16	21	20	18	23	17
9	23	22	19	22	22	21	22	20
13	19	20	16	22	21	17		
14	24	26	23	25	26	24	27	24
15	22	25	20	25	24	21	26	21
19	25	25	23	24	25	23	27	24
20	17			19	18	17	21	19
29 [1]		26	22	27	31	25	32	24
30	20	24	21	25	20	19	23	20
32	23	24	22	23	26	21	26	21
33	23	23	20	22	25	20	25	21
34	22	21	15	23	23	19	26	22
35	23	19	17	22	24	22	24	22
37	19	19	17	20	20	17	18	14
40	24	26	18	26	27	21	27	18

[1] Values listed under the 60–70 grade are for the 40–50 penetration grade.

tration at 50° F. changes the value of the slope in the second significant figure and thus, since this is within the experimental error, only two significant figures are reported. It is seen from table 9 that in every case the residue from the thin-film oven test has a lower slope and thus has become less susceptible to temperature changes than the original asphalt. This result indicates that some oxidization takes place during the test.

There is no definite relation between the susceptibilities of the original asphalts and the ductilities or penetrations of the residues from the thin-film oven test except that the asphalts of low susceptibility consistently fail to meet the proposed ductility requirements. However, the converse is not true since the residues from a number of the asphalts of high susceptibility also fail to pass the proposed ductility requirements. Table 9 gives the slopes of the log penetration-temperature curves for 81 samples of original asphalt. The slopes for 20 of these samples are 21×10^{-3} or less and the residues from all these materials failed to meet the ductility requirements. The remaining 61 samples of higher susceptibility are divided about equally between negative-spot and positive-spot materials, there being 32 of the former and 29 of the latter. The residues from 7 of the 32 negative-spot materials and 18 of the 29 positive-spot materials failed to meet the ductility requirements. The residues from 2 of the 11 positive-spot materials that met the ductility requirements failed to meet the proposed penetration requirements.

CONCLUSIONS

The results of the tests described in this report justify the following conclusions:

1. For asphalts manufactured by the same refining process from the same base petroleum the relation of the ductility to penetration is the same for all penetration grades regardless of the temperature at which a given penetration occurs.

2. The ductilities of the residues from the thin-film oven test show a common relation to penetration that is different from the relation for the original asphalt, the ductility of the residue being generally less than that of the original sample when the comparison is made at the same penetration.

3. A requirement that the sample shall have a loss in weight not greater than 1 percent when heated in an ⅛-inch film for 5 hours at 325° F. appears to be suitable for all penetration grades of asphalt from the 50–60 through the 120–150 grades.

4. A requirement that the residue from the thin-film oven test (⅛-inch film heated 5 hours at 325° F.) shall have a penetration at 77° F. (100 gram, 5 seconds) of at least 50 percent of that of the original sample appears to be equally suitable for all penetration grades of asphalt from the 50–60 through the 120–150 grades.

5. The requirement that the residue from the thin-film oven test (⅛-inch film heated 5 hours at 325° F.) shall have a ductility at 77° F. (5 centimeters per minute) of not less than 40 centimeters for the 50–60 and 60–70 penetration grades, and not less than 100 centimeters for the 85–100, 100–120, and 120–150 penetration grades gives comparable results for all these grades.

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